WSDOT PAVEMENT GUIDE

Volume 1

Pavement Policy

Washington State Department of Transportation

Environmental and Engineering Programs Division
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TABLE OF CONTENTS

1. PURPOSE

1.1 GENERAL

This guide is the product of WSDOT pavement experience, research (state, national, international) and various analyses. The need for this document became more apparent as WSDOT adopted new state-of-the-practice and state-of-the-art design practices. Further, the manual will be of value to both Region and Headquarters (HQ) personnel in designing and evaluating pavement structures.

1.2 RELATIONSHIP TO WSDOT DESIGN MANUAL

Pavement design information previously contained in the Design Manual is largely replaced by this guide. Refer to Division 5 in the Design Manual for additional pavement related information.

2. INTRODUCTION

2.1 BASIC ELEMENTS

The basic elements of the paving structure include the surfacing, base course (stabilized or unstabilized) and subbase course(s) (as required). Pavement structures are divided into two general classifications based on the type of pavement structure typical to each: flexible and rigid. Flexible pavements have some type of bituminous surfacing and rigid pavements have a surfacing of Portland cement concrete (PCC).

2.2 PURPOSE OF SURFACING AND BASE COURSES

The surfacing and base courses are layers of high stiffness and density. Their principal purpose is to distribute the wheel load stresses within the pavement structure and thus protect the subgrade soils against excessive deformation or displacement. Where water is expected to accumulate, the base material shall be free-draining.

2.3 FROST ACTION

Greater depths of base or selected free-draining borrow materials are usually necessary in areas where frost action is severe or the subgrade soil is extremely weak. The total depth of the pavement structure is extremely important in high frost penetration areas. Additional thickness of non-frost susceptible base or subbase materials is often effectively used to combat this problem. An effective measure is to have the pavement structure (total of surfacing and base) at least equal to one-half the maximum expected depth of freeze when the subgrade is classified as a frost susceptible soil.

3. PAVEMENT DESIGN CONSIDERATIONS

3.1 DESIGN PERIOD

The design period is the time from original construction to a terminal condition for a pavement structure. AASHTO essentially defines design period, design life and performance period as being the same terms. AASHTO defines an analysis period as the time for which an economic analysis is to be conducted. Further, the analysis period can include provisions for periodic surface renewal or rehabilitation strategies which will extend the overall service life of a pavement structure before complete reconstruction is required.

The design periods used by WSDOT are chosen so that the design period traffic will result in a pavement structure sufficient to survive through the analysis period. It is recognized that intermittent treatments may be needed to preserve the surface quality and ensure that the structure lasts through the analysis period. The required design periods are shown in Table 1:

Table 1. Required Design Period

Highway Description	Design Period
Interstate and Principal Arterial	50 years
Minor Arterial, Collector with ESALs greater than 100,000 per year	50 years
Minor Arterial, Collector with ESALs less than 100,000 per year	20 years

The 50 year design periods can be reduced for unique, project specific conditions such as temporary HOV lane pavements, future planned realignment or grade changes, etc. The 20 year design period can be increased for those routes with future, large expected increases in traffic (ESALs), anticipated functional class changes, etc.

It should be noted that doubling the design period traffic (ESALs) adds about one inch of HMA or PCC to the initial structural thickness of a flexible or rigid pavement design.

3.2 TRAFFIC

The volume and character of traffic, expressed in terms of 18,000 lbs equivalent single axle loads (ESALs) for structural design purposes strongly influences pavement structural requirements. Both flexible and rigid pavement structures can be designed to meet most ESAL requirements; however, this does not imply similar maintenance and resurfacing requirements.

3.3 SUBGRADE SOILS

The characteristics of native soils directly affect the pavement structure design. A careful evaluation of soil characteristics is a basic requirement for each individual pavement structure design.

Resilient modulus is the primary material inputs into the *AASHTO Guide for Design of Pavement Structures (1993)*, as well as the WSDOT HMA overlay design procedure. WSDOT pavement designs shall be based on the use of the materials resilient modulus.

4. PAVEMENT TYPE SELECTION

There are three primary areas that need to be addressed; pavement design analysis, life cycle cost analysis, and engineering analysis. Each of these areas can have a significant impact on the selected pavement type and will be further described in this protocol. The overall process is shown in Figure 1.

The pavement type selection protocol is applicable to all new alignment, ramps, collector-distributors, acceleration-deceleration lanes, and existing pavement reconstruction on interstate, principal arterials, and any other roadway that may benefit from this analysis. Pavement type selection is not necessary for chip seal roadways. For mainline widening, if the selected pavement type is the same pavement type as the existing, then a pavement type selection is not required. When comparing life cycle costs of the different alternatives, the comparison must be based on the total costs, which include initial construction, maintenance, rehabilitation, and user costs.

4.1 APPLICATION OF PAVEMENT TYPE SELECTION

The following is a list of considerations for new construction or reconstruction of mainline, ramps, collector-distributors, acceleration-deceleration lanes, and intersections.

- **Mainline lane reconstruction.** A pavement type selection must be completed on all mainline pavements that are more than ½ lane mile in length or more than $0.5 million. For roadway segments shorter in length or lower in cost, the HQ Materials Laboratory – Pavements Division should be contacted for further direction on the need to conduct a pavement type selection.
- **Ramps.** For ramps with mature (lane configuration or right of way limits the expansion of the roadway footprint) geometrics, high traffic and high truck percentages both PCC and HMA pavement should be considered.
- **Collector-Distributors.** Collector-distributors should be designed similarly as ramps above.
- **Acceleration-Deceleration Lanes.** Treat the same as collector-distributors.
- **Intersections.** Most intersections will not require an analysis separate from the rest of the highway. However, intersections with chronic, relatively short term, rutting should be examined in detail to determine the nature and cause of the rutting and to carefully consider alternate pavement types. The HQ Materials Laboratory – Pavements Division, should be contacted for further guidance and direction on options for resolving chronic intersections rutting issues.

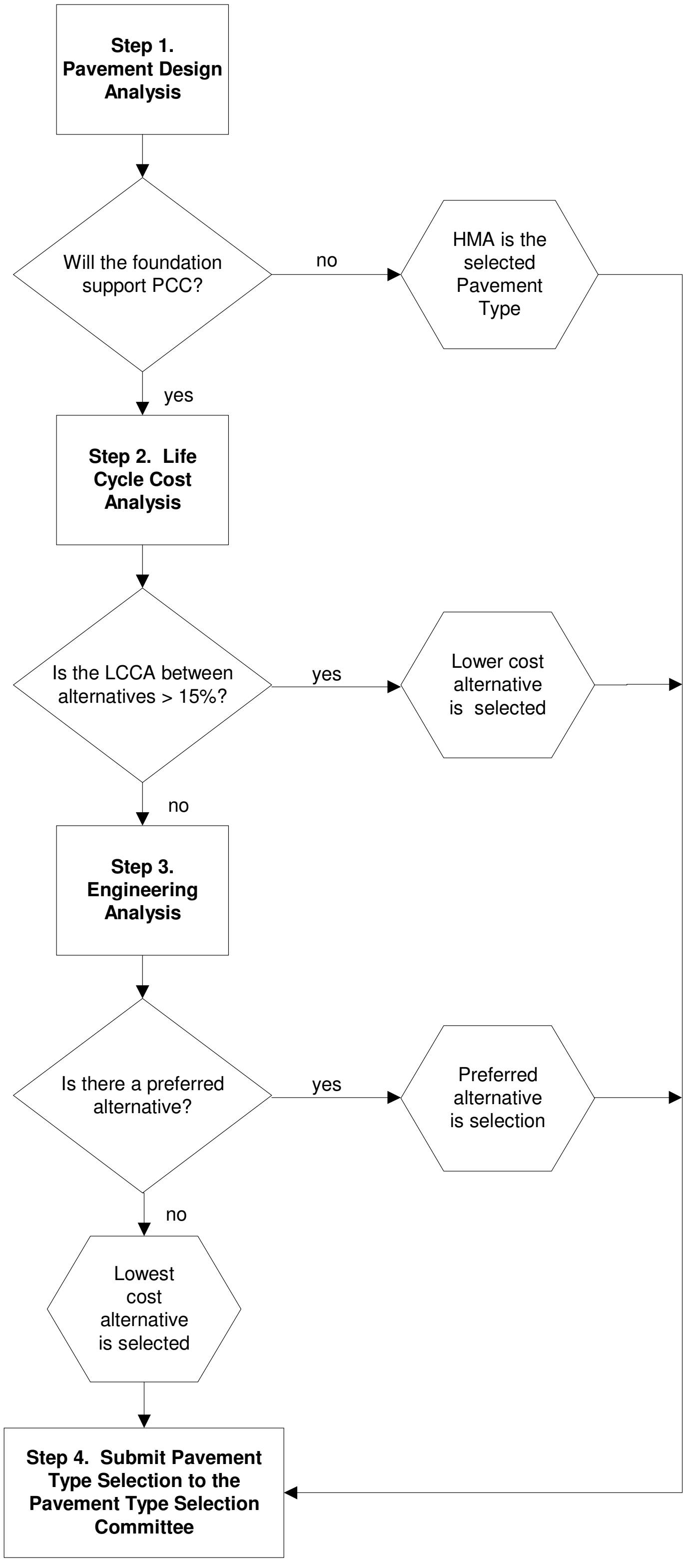


Figure 1. Pavement Type Selection Flow chart

4.2 PAVEMENT DESIGN ANALYSIS

The pavement design should be performed first, since the results may preclude the need to continue with the remainder of the pavement type selection protocol process (life cycle cost analysis and engineering analysis).

The pavement design analysis should include the review and analysis of the following: subgrade competency, traffic analysis, materials, climate/drainage, environment, construction considerations, and any other pavement design factors.

4.2.1 SUBGRADE COMPETENCY

This is the only "go/no go" decision to be made under the Pavement Design Analysis. HMA tends to perform better in situations where long-term settlement is expected, due to simplified patching and overlay opportunities compared to PCC pavement. In such cases, the pavement design analysis might preclude the need to continue with a life cycle cost analysis and the engineering analysis if subgrade conditions will not allow the use of PCC pavement. If the engineering evaluation of the subgrade concludes that PCC pavement cannot be placed or will not perform successfully, then the pavement type selection is complete and HMA is the selected pavement type. If the engineering evaluation of the subgrade concludes that either pavement type could be used successfully, proceed with the remainder of the pavement type selection process.

4.2.2 CLASSIFICATION FOR PAVEMENT DESIGN

Pavements can be divided into different traffic classes depending on extremely light to extremely heavy traffic. Flexible and rigid pavements can be designed to accommodate these wide traffic ranges. For each of the pavement classes, traffic can be quantified by the number of standard axles/lane (or load spectra) or by traffic volume (e.g., average daily traffic (ADT), annual average daily traffic (AADT), annual average daily truck traffic (AADTT), or percent trucks). Based on the traffic volume and traffic growth rate, the design traffic loading can be estimated over the structural design period or the analysis period. The design traffic loading determines the pavement thickness needed to support the traffic loading over the structural design period.

Correctly estimating design traffic is crucial to selecting an appropriate pavement. To calculate the total design traffic per lane that a pavement will carry over its structural design life, it is necessary to estimate present traffic loading. To estimate future traffic loadings, traffic growth rates should be used. Depending on the roadway segment's importance, conducting a sensitivity analysis to compare growth rates and the impact of the growth rate on pavement thickness may be worthwhile.

4.2.3 MATERIALS

Selecting materials for a road pavement design is determined by the availability of suitable materials, environmental considerations, construction methods, economics, and previous performance. To select the materials that best suit the conditions, these factors must be evaluated during the design to ensure a whole-life cycle strategy.

4.2.3.1 Availability and Performance

Most road construction materials have been classified and specifications prepared for each of the material classes. Every road pavement, independent of its type and applied materials, is subjected to certain traffic loads and environmental factors. These factors create various deterioration modes under in-service conditions. Deterioration modes and the pavement's susceptibility to various deteriorating factors depend on the type of pavement and materials applied. Table 2 shows the pavement deterioration modes for HMA and PCC pavements.

Table 2. Pavement Deterioration Modes

HMA Pavements	**PCC Pavements**
▪ Surface deterioration – Decrease in friction – Rutting – Surface cracking – Raveling (stripping) – Roughness – Studded tire wear ▪ Structural deterioration – Base and subgrade rutting – Fatigue cracking – Reflective cracking	▪ Surface deterioration – Decrease in friction – Surface cracking – Curling and warping – Joint raveling – Roughness – Studded tire wear ▪ Structural deterioration – Cracking – Pumping – Faulting

Pavement surface defects may only require surface course maintenance or rehabilitation. Structural deterioration is a defect of the whole pavement structure and treating it may require more extensive pavement rehabilitation. Knowing the difference between these two types of deterioration is important to maintaining and properly understanding pavement durability (or pavement life).

Past performance with a particular material should be considered in tandem with applicable traffic and environmental factors. The performance of similar pavements or materials under similar circumstances should also be considered. Information from pre-existing designs, material tests, and pavement management data can help characterize a specific material's suitability for pavement applications.

WSDOT's experience has been that all pavement types are affected by studded tire wear (see Figures 2 and 3). The abrasion on pavement surfaces caused by studded tires, wears down the pavement surface at a much greater rate then any other pavement/tire interaction. Surface deterioration has occurred in as little as 4 to 6 years on HMA and 10 to 15 years on PCC pavements. For the pavement type selection process, this implies that future rehabilitation timing may be reduced for each pavement type due to the damaging effect of studded tires and should be considered in the analysis until such a time that studded tire use is prohibited.

4.2.3.2 Recycling

To enhance sustainable development, consider using recycled materials in roadway construction. Future rehabilitation or maintenance treatments, if applicable, should incorporate recycled materials whenever possible.

Figure 2. Studded Tire Wear on PCC

Figure 3. Studded Tire Wear on HMA

4.2.4 Climate/Drainage

Both surface runoff and subsurface water control must be considered. Effective drainage design prevents the pavement structure from becoming saturated. Effective drainage is essential for proper pavement performance and is assumed in the structural design procedure.

4.2.5 Pavement Design

Pavement design shall be conducted in accordance with the *AASHTO Guide for Design of Pavement Structures – 1993* and this Guide. All pavement designs, rehabilitation strategies, and rehabilitation timing must be submitted, for approval, to the Pavement Design Engineer at the HQ Materials Laboratory.

Input values to be used in the *AASHTO Guide for Design of Pavement Structures – 1993* are shown in Table 3. Additional information can be obtained from the WSDOT Electronic Pavement Guide[1].

[1] http://wwwi.wsdot.wa.gov/maintops/mats/Apps/Pavement%20Guide%20Interactive/index.htm or a copy can be purchased from Engineering Publications at http://www.wsdot.wa.gov/fasc/EngineeringPublications/order.htm

Table 3. WSDOT Inputs for AASHTO Pavement Design

Input Parameter	HMA	PCC
ΔPSI	1.5	1.5
a_{HMA}	0.44	N/A
a_{CS}	0.13	N/A
$a_{HMA\ base}$	0.44	N/A
C_d	N/A	1.0
E_c	N/A	4,000,000 psi
J (dowels)	N/A	2.7
J (no dowels)	N/A	3.4
k_{CSBC}	N/A	200 pci
$k_{HMA\ base}$	N/A	400 pci
m	1.0	N/A
M_R (average soil)	10,000 psi	N/A
M_R (good soil)	20,000 psi	N/A
M_R (poor soil)	5,000 psi	N/A
S_c'	N/A	650 psi
S_o	0.50	0.40

4.2.5.1 Additional PCC Issues

WSDOT has demonstrated that the PCC pavements constructed in the late 1950's to early 1960's are able to obtain a 50-year or more pavement life as long as joint faulting can be overcome. The ability to provide adequate joint design to minimize joint faulting is addressed by requiring the use of non-erodable bases and dowel bars (1-½ inch diameter by 18 inch length) at every transverse joint. The use of epoxy-coated dowel bars, both locally and nationally, does not necessarily ensure that a 50-year performance life will be obtained. Several states have observed that the corrosion of epoxy coated dowel bars occurs within 15 to 20 years. Minnesota Department of Transportation has conducted a study of in-service concrete pavements that were constructed with epoxy-coated steel dowel bars at transverse joints and has determined that significant corrosion has occurred in the dowel bars. The result of this study has indicated that the corrosion of epoxy coated dowel bars results in a pavement life of less than 20 years (dowel bar corrosion leads to joint deterioration which requires either complete replacement of the concrete pavement or a dowel bar retrofit). California Department of Transportation has conducted a study on the corrosion rates of a variety of different dowel bars (epoxy coated, solid stainless steel and stainless steel clad) and has found that the epoxy-coated bars failed the corrosion testing, while the stainless steel bars (clad and solid) experienced no corrosion. Therefore, WSDOT currently requires the use of stainless steel clad dowel bars on all newly constructed concrete pavements (also see APPENDIX 1) in western Washington and mountain passes and corrosion resistant alternatives for eastern Washington. The anticipated life of a PCC pavement shall be on the order of 50 years. Rehabilitation of these PCC pavements will potentially require diamond grinding at the 20 to 30 year range to address studded tire wear.

4.2.5.2 Additional HMA Issues

For heavily trafficked roadways (primarily the interstate and principal arterials), the pavement thickness should be designed to such a depth that future roadway reconstruction is not necessary. The pavement thickness should be designed such that 50 years of traffic will not generate significant bottom up (fatigue) cracking. Future mill and fill or HMA overlay will be required to address surface distress (rutting or top down cracking) and aging of the HMA surface.

4.2.5.3 Effect of Studded Tire Wear

WSDOT is currently in the process of investigating a number of mitigation techniques for the wear that results on PCC pavements due to studded tires. These include increasing the PCC flexural strength and utilization of a combined aggregate gradation. At this time, both of these studies are still in progress and conclusions are yet to be drawn. Therefore, to combat the damaging effects of studded tires, it is recommended that the PCC thickness be increased by one inch to accommodate future diamond grinding(s). This damage is also a concern for HMA pavements. WSDOT has constructed a number of stone matrix asphalt (SMA) pavements, but have had a number of construction related difficulties, such that the ability to determine the impact that a SMA will have on reducing studded tire damage has yet to be determined. In the life cycle cost analysis, the accelerated wear on HMA pavements will be incorporated through a shorter performance period on future overlays (as supported by Pavement Management data).

4.2.6 CONSTRUCTION CONSIDERATIONS

Pavement construction issues are an important component of the selection of pavement type. These issues can include:

- Pavement thickness constraints. Consider the impact of utilities below the pavement and overhead clearances may have on limiting the layer thickness and type, and/or limit future overlay thickness.
- Effects on detours, bypasses, and alternate routes. Consider the geometric and structural capacity of detours, bypasses and alternate routes to accommodate rerouted traffic.
- Effects of underground pipes and services on performance. Determine the impact of existing utilities and future utility upgrades on initial and future rehabilitation treatments.
- Anticipated future improvements and upgrades. Consider if the pavement type restricts or minimizes the ability to efficiently and cost effectively upgrade and/or improve the roadway width, geometry, structural support, etc.
- Impact on maintenance operations, including winter maintenance. Will the selected pavement type have impacts due to freeze-thaw (surface and full-depth) or snow and ice removal?
- Grades, curvature, and unique loadings (slow-moving vehicles and starting and stopping). How will steep grades, curvature and unique loadings impact pavement performance? Slow moving vehicles will generate increased strain levels in the HMA pavement structure and these strains can significantly impact pavement performance (i.e. rutting and cracking).

- A schedule analysis may need to be conducted to determine critical construction features (haul truck access, traffic control constraints – road closures, etc) and their impact on the project. This should also include staging analysis for multiple projects within the project corridor (to ensure that alternate routes are free of traffic delay due to construction activities). The Construction Analysis for Pavement Rehabilitation Strategies CA4PRS[2] software is useful in determining construction impacts and duration.

4.2.7 OTHER FACTORS

Evaluate other factors that are unique to the project or corridor.

4.3 LIFE CYCLE COST ANALYSIS

Life cycle cost analysis provides a useful tool to assist in the pavement type selection. Only differential factors should be considered. The alternative resulting in the lowest net present value or annualized cost over a given analysis period is considered the most cost efficient.

Life cycle costs refer to all costs that are involved with the construction, maintenance, rehabilitation and associated user impacts of a pavement over a given analysis period. Life cycle cost analysis is an economic comparison of all feasible construction or rehabilitation alternatives, evaluated over the same analysis period. "A feasible alternative is one that fits with the required constraints (e.g., geometric, construction time, traffic flow conditions, clearances, right-of-way, maximum funds available, etc.)" [1]. At a minimum, one HMA and one PCC alternative should be evaluated. The total cost (initial construction, maintenance, rehabilitation, and user costs) of each design alternative can be compared based on the present value or equivalent uniform annual cost.

The life cycle cost analysis should be conducted using the FHWA life cycle cost analysis software, which is available through the HQ Materials Laboratory – Pavements Division[3].

The Federal Highway Administration's policy[4] on life cycle cost analysis "is that it is a decision support tool, and the results of the life cycle cost analysis are not decisions in and of themselves. The logical analytical evaluation framework that life cycle cost analysis fosters is as important as the life cycle cost analysis results themselves." [4].

Net present value is the economic efficiency indictor of choice [4]. The annualized method is appropriate, but should be derived from the net present value. Computation of benefit/cost ratios is generally not recommended because of the difficulty in sorting out costs and benefits for use in the benefit/cost ratios [4].

[2] The CA4PRS software can be downloaded at http://wwwi.wsdot.wa.gov/maintops/mats/apps/CA4PRS.htm.
[3] http://wwwi.wsdot.wa.gov/MaintOps/mats/apps/LCCA.htm
[4] Federal Highway Administration, Final Policy Statement on LCCA published in the September 18, 1996, *Federal Register*.

Future costs should be estimated in constant dollars and discounted to the present using a discount rate. The use of constant dollars and discount rates eliminates the need to include an inflation factor for future costs.

4.3.1 NET PRESENT VALUE

The present value method is an economic method that involves the conversion of all of the present and future expenses to a base of today's costs [2]. The totals of the present value costs are then compared one with another. The general form of the present value equation is as follows:

$$NPV = F \frac{1}{(1+i)^n}$$

where,

NPV = Net Present Value
F = Future sum of money at the end of n years
n = Number of years
i = Discount rate

4.3.2 ANNUALIZED METHOD

The annualized method is an economic procedure that requires converting all of the present and future expenditures to a uniform annual cost [2]. This method reduces each alternative to a common base of a uniform annual cost. The costs are equated into uniform annual costs through the use of an appropriate discount rate [3]. Recurring costs, such as annual maintenance, are already expressed as annual costs. A given future expenditure, such as a pavement overlay, must first be converted to its present value before calculating its annualized cost. The general form of the Annualized cost equation is as follows:

$$A = PV \frac{i(1+i)^n}{(1+i)^n - 1}$$

where,

A = Annual cost
PV = Present Value
n = Number of years
i = Discount rate

4.3.3 ECONOMIC ANALYSIS

The costs to be included in the analysis are those incurred to plan, work on and maintain the pavement during its useful life. All costs that can be attributed to the alternative and that differ from one alternative to another must be taken into account. These include costs to the highway agencies and user costs.

4.3.3.1 Performance Period

As a pavement ages, its condition gradually deteriorates to the point where some type of rehabilitation treatment is necessary. The timing between rehabilitation treatments is defined as the performance life. An example of this is illustrated in Figure 4. Performance life for the initial pavement design and subsequent rehabilitation activities has a major impact on life cycle cost analysis results [4].

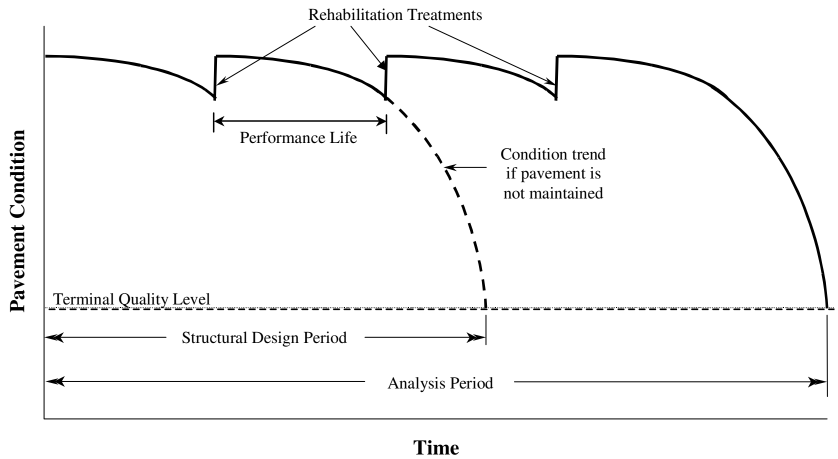


Figure 4. Example of Pavement Performance Life

When available, the performance life of the various rehabilitation alternatives should be determined based on past performance history. In these cases, the Washington State Pavement Management System (WSPMS) provides history on past pavement performance lives. In instances where the anticipated performance life is not well established (i.e., due to improved engineering and technologies), selection of the performance life should be coordinated and concurred upon with the HQ Materials Laboratory – Pavements Division.

4.3.3.2 Initial Construction Costs

Unit costs vary according to location, the availability of materials, the scope of the project, and the standards to be complied with. They can be estimated based on previous experiences, generally by averaging the bids submitted for recent projects of similar scope. Typical item costs can be located in bid item tabulations. The bid item costs may need to be adjusted according to local availability and work constraints. Mobilization, engineering and contingencies, and preliminary engineering can be excluded (sales tax should be included) for the initial construction cost estimate, since these costs are similar for HMA and PCC.

4.3.3.3 Maintenance and Rehabilitation Costs

The type and frequency of future maintenance and rehabilitation operations vary according to the pavement type being considered. Knowing how a particular pavement type performed in the past is a valuable guide in predicting future performance [5]. The WSPMS should be reviewed for past performance of rehabilitation and maintenance schedules. Costs must always be determined as realistically and accurately as possible based on local context and specific project features.

When calculating the total construction cost, include the cost of pavement resurfacing or PCC rehabilitation, milling or diamond grinding, shoulders, pavement repair, drainage and guardrail adjustments, maintenance and protection of traffic, etc. Mobilization (5%), engineering and contingencies (15%), preliminary engineering (10%), and sales tax should be included in all rehabilitation costs.

Construction duration should reflect the actual construction time that is required for each pavement type. Construction durations should consider improvements, proposals or innovative contracting procedures in construction processes.

If a difference exists in routine maintenance costs between the various alternatives, these costs should be included in the life cycle cost analysis.

Table 4 contains a probable scenario corresponding to average traffic and climate conditions, assuming that state-of-the-art practices have been followed during construction and that maintenance and rehabilitation projects are carried out efficiently and on schedule.

Table 4. Rehabilitation Scenario for HMA and PCC Pavements

Year	HMA Pavement	PCC Pavement
0	Construction or reconstruction	Construction or reconstruction
15	0.15’ mill and HMA overlay	
20		Diamond grinding
30	0.15’ HMA overlay	
40		Diamond grinding
45	0.15’ mill and HMA overlay	
50	Salvage value (if applicable)	Salvage value (if applicable)

4.3.3.4 Salvage Value

Salvage value is the asset value at the end of the analysis period. The difference between the salvage values of the various alternatives for a project can be small, because discounting can considerably reduce this value, but the size of this reduction is influenced by the actual discount rate chosen. As for the value assigned to the pavement materials, or terminal value, predicting the proportion of recovery or recycling of these materials on-site at the end of the analysis period is uncertain.

If an alternative has reached its full life cycle at the end of the analysis period, it is generally considered to have no remaining salvage value. If it has not completed a life cycle, it is given a salvage value, which is usually determined by multiplying the last construction or rehabilitation cost, by the ratio of the remaining expected life cycle to the total expected life.

$$\text{Salvage Value} = \text{CC x } \frac{\text{ERL}}{\text{TEL}}$$

where,

CC = Last construction or rehabilitation project costs
ERL = Expected remaining life of the last construction or rehabilitation project
TEL = Total expected life of the last construction or rehabilitation project

4.3.3.5 User Costs

It is difficult to determine whether or not one rehabilitation alternative results in a higher vehicle operating cost than another. Therefore, the user costs associated with each of the rehabilitation alternatives shall be determined using only costs associated with user delay. This shall be based on the construction periods and the traffic volumes that are affected by each of the rehabilitation alternatives.

Several studies have been performed that associate cost with the amount of time the user is delayed through a construction project. The method used is not as important as using the same method for each of the alternatives.

The costs associated with user delays are estimated only if the effects on traffic differ among the alternatives being analyzed. For future rehabilitation work, user costs associated with delays can be substantial for heavily travelled roadways, especially when work is frequent.

While there are several different sources for the dollar value of time delay, the recommended mean values and ranges for the value of time (in August 1996 dollars) shown in Table 5, are reasonable.

Table 5. Recommended Dollar Values per Vehicle Hour of Delay [4] (adjusted to 2004 dollars)[5]

Vehicle Class	Value Per Vehicle Hour	
	Value	**Range**
Passenger Vehicles	$13.96	$12 to $16
Single-Unit Trucks	$22.34	$20 to $24
Combination Trucks	$26.89	$25 to $29

4.3.3.6 Other Costs

Surfacing types and characteristics influence the noise emitted on tire-to-pavement contact. If construction of a noise attenuation structure is planned, the cost of that structure must be included in the treatment costs of the alternative being analyzed. The issue of safety can be addressed similarly.

[5] Calculator for converting costs to current dollars can be accessed at http://data.bls.gov/cgi-bin/cpicalc.pl

4.3.3.7 Discount Rate

"In a life cycle cost analysis, a discount rate is needed to compare costs occurring at different points in time. The discount rate reduces the impact of future costs on the analysis, reflecting the fact that money has a time value" [6]. The discount rate is defined as the difference between the market interest rate and inflation, using constant dollars.

Table 6 shows recent trends in the real treasury interest rates for various analysis periods published in the annual updates to OMB Circular A-94 [7].

Table 6. Real Treasury Interest Rates [7]

Year	3-Year	5-Year	7-Year	10-Year	30-Year
1979	2.8	3.4	4.1	4.6	5.4
1980	2.1	2.4	2.9	3.3	3.7
1981	3.6	3.9	4.3	4.4	4.8
1982	6.1	7.1	7.5	7.8	7.9
1983	4.2	4.7	5.0	5.3	5.6
1984	5.0	5.4	5.7	6.1	6.4
1985	5.9	6.5	6.8	7.1	7.4
1986	4.6	5.1	5.6	5.9	6.7
1987	2.8	3.1	3.5	3.8	4.4
1988	3.5	4.2	4.7	5.1	5.6
1989	4.1	4.8	5.3	5.8	6.1
1990	3.2	3.6	3.9	4.2	4.6
1991	3.2	3.5	3.7	3.9	4.2
1992	2.7	3.1	3.3	3.6	3.8
1993	3.1	3.6	3.9	4.3	4.5
1994	2.1	2.3	2.5	2.7	2.8
1995	4.2	4.5	4.6	4.8	4.9
1996	2.6	2.7	2.8	2.8	3.0
1997	3.2	3.3	3.4	3.5	3.6
1998	3.4	3.5	3.5	3.6	3.8
1999	2.6	2.7	2.7	2.7	2.9
2000	3.8	3.9	4.0	4.0	4.2
2001	3.2	3.2	3.2	3.2	3.2
2002	2.1	2.8	3.0	3.1	3.9
2003	1.6	1.9	2.2	2.5	3.2
Average	3.4	3.8	4.1	4.3	4.7
Std Dev	1.1	1.2	1.3	1.4	1.4

For all life cycle cost analysis, WSDOT has selected a discount rate of four percent.

4.3.3.8 Analysis Period

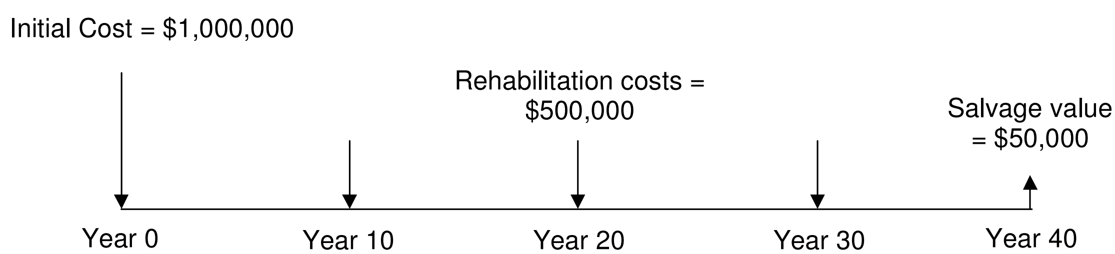
The analysis period is the time period used for comparing design alternatives. An analysis period may contain several maintenance and rehabilitation activities during the life cycle of the pavement being evaluated [6]. In general, the recommended analysis period coincides with the useful life of the most durable alternative. "In the past, pavements were typically designed and analyzed for a 20 year performance period, since the original Interstate Highway Act in 1956 required that traffic be considered through 1976. It is now recommended that consideration be given to longer analysis periods, since these may be better suited for the evaluation of alternative long term strategies based on life cycle costs. Consideration should be give to extending the analysis period to include one rehabilitation" [6]. Table 7 contains WSDOT recommended analysis periods.

Table 7. WSDOT Recommended Analysis Periods by Traffic Level

Traffic Level	**Analysis Period (years)**
Interstate or Principal Arterial	50
Minor Arterial or Major Collector	20

4.3.3.9 Risk Analysis

The deterministic approach to life cycle costs involves the selection of discrete input values for the initial construction costs, routine maintenance and rehabilitation costs, the timing of each of these costs, and the discount rate. These values are then used to calculate a discrete single value for the present value of the specified project. The deterministic approach applies procedures and techniques without regard for the variability of inputs. An example of the deterministic approach is shown in below.



Discount rate = 4 percent

$$PW = \$1{,}000{,}000 + \frac{\$500{,}000}{(1.04)^{10}} + \frac{\$500{,}000}{(1.04)^{20}} + \frac{\$500{,}000}{(1.04)^{30}} + \frac{\$500{,}000}{(1.04)^{40}} - \$50{,}000$$

$$= \$1{,}709{,}720$$

The deterministic approach is a viable method for determining life cycle costs; however, life cycle cost analysis contains several possible sources of uncertainty. In certain cases, the uncertainty factors may be sizeable enough to affect the ranking of the alternatives. To obtain more credible results, a systematic evaluation of risk should always be carried out. The primary disadvantage of the deterministic approach is that it does not account for the input parameter variability.

The concept of risk comes from the uncertainty associated with future events – the inability to know what the future will bring in response to a given action today [4]. Risk analysis is concerned with three basic questions [4]:

1. What can happen?
2. How likely is it to happen?
3. What are the consequences of it happening?

Risk analysis answers these questions by combining probabilistic descriptions of uncertain input parameters with computer simulation to characterize the risk associated with future outcomes [4]. It exposes areas of uncertainty typically hidden in the traditional deterministic approach to life cycle cost analysis, and it allows the decision maker to weigh the probability of an outcome actually occurring [4].

The two most commonly used methods of assessing the risk are probabilistic analysis and sensitivity analysis. The probabilistic approach combines probability descriptions of analysis inputs to generate the entire range of outcomes as well as the likelihood of occurrence. Probabilistic analysis represents uncertainties more realistically than does a sensitivity analysis. Sensitivity analysis assigns the same weighting to all extreme or mean values, whereas probabilistic analysis assigns the lowest probability to extreme values. A probabilistic analysis is advocated, but if this is not possible, a sensitivity analysis at the very least should be carried out.

4.3.3.10 Probabilistic Analysis

The probabilistic approach takes into account the uncertainty of the variables used as inputs in the life cycle cost analysis. The probability distribution is selected for each input variable, which are then used to generate the entire range of outcomes and the likelihood of occurrences for both the associated costs and the performance life. The procedure often used to apply a probability distribution is a "Monte Carlo Simulation". The Monte Carlo Simulation is a computerized procedure that takes each input variable, assigns a range of values (using the mean and standard deviation of the input variable), and runs multiple combinations of all inputs and ranges to generate a life cycle cost probability distribution. Using the probabilistic approach allows for the ability of determining the variability or "spread" of the life cycle cost distributions and determining which alternative has the lower associated risk (see Figure 5).

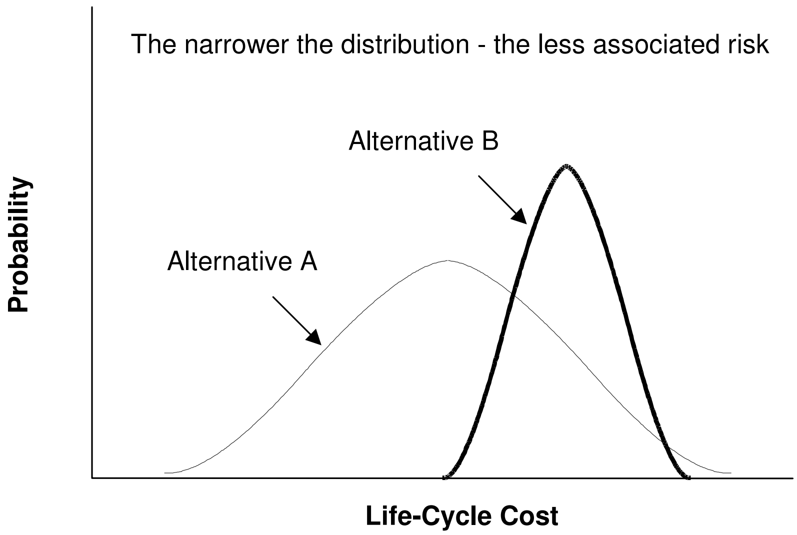


Figure 5. Probability Distribution

An example of a probabilistic analysis is included in APPENDIX 2. WSDOT input values for the probabilistic analysis are contained in APPENDIX 3.

By performing the Monte Carlo computer simulation, thousands, even tens of thousands of samples are randomly drawn from each input distribution to calculate a separate what-if scenario [4]. Risk analysis results are presented in the form of a probability distribution that describes the range of possible outcomes along with a probability weighting of occurrence [4]. With this information, the decision maker knows not only the full range of possible values, but also the relative probability of any particular outcome actually occurring [4].

4.3.3.11 Sensitivity Analysis

Sensitivity analysis is a technique used to determine the influence of major input assumptions, projections, and estimates on life cycle cost analysis results. In a sensitivity analysis, major input values are varied (either within some percentage of the initial value or over a range of values) while all other input values remain constant and the amount of change in results is noted [4].

An example of a sensitivity analysis is shown below.

> Two pavement design strategies with discount rates that vary from two to six percent over a 35-year analysis period will be described.
>
> Figure 6 summarizes Table 8 and Table 9 show the comparison of net present value at the various discount rates. For this example, Alternative 1 is more expensive at discount rates of five percent and lower, while Alternative 2 is more expensive at discount rates six percent and above.

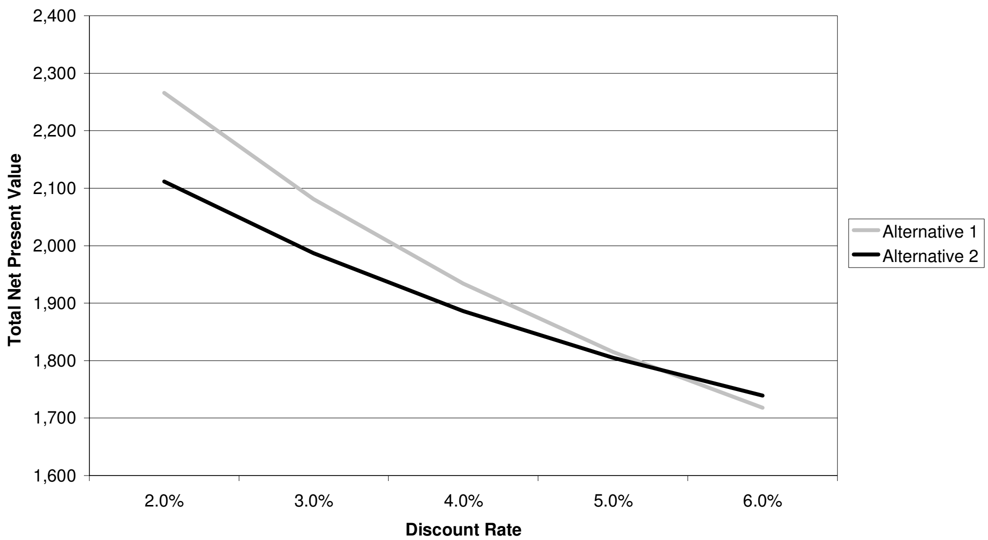


Figure 6. Sensitivity of Net Present Value to Discount Rate

Table 8. Sensitivity Analysis – Alternative 1 [4]

Activity	Year	Cost	Net Present Value				
			2.0%	**3.0%**	**4.0%**	**5.0%**	**6.0%**
Construction	0	975	975	975	975	975	975
User Cost	0	200	200	200	200	200	200
Rehab #1	10	200	164	149	135	123	112
User Cost #1	10	269	220	200	182	165	150
Rehab #2	20	200	135	111	91	75	62
User Cost #2	20	361	243	200	165	136	113
Rehab #3	30	200	110	82	62	46	35
User Cost #3	30	485	268	200	150	112	85
Salvage	35	-100	-50	-36	-25	-18	-13
TOTAL NPV			2,266	2,081	1,934	1,815	1,718

Table 9. Sensitivity Analysis – Alternative 2 [4]

Activity	Year	Cost	Net Present Value				
			2.0%	**3.0%**	**4.0%**	**5.0%**	**6.0%**
Construction	0	1,100	1,100	1,100	1,100	1,100	1,100
User Cost	0	300	300	300	300	300	300
Rehab #1	15	325	241	209	180	156	136
User Cost #1	15	269	200	173	139	129	112
Rehab #2	30	325	179	134	100	75	57
User Cost #2	30	361	199	149	111	84	63
Salvage	35	-217	-108	-77	-55	-39	-28
TOTAL NPV			2,112	1,987	1,886	1,805	1,739

A primary drawback of the sensitivity analysis is that the analysis gives equal weight to any input value assumptions, regardless of the likelihood of occurring [4]. In other words, the extreme values (best case and worst case) are given the same likelihood of occurrence as the expected value, which is not realistic [4].

4.4 ENGINEERING ANALYSIS

After completing the pavement design analysis and the life cycle cost analysis, the engineering analysis is conducted when there are two viable alternatives. Finding the HMA and PCC alternatives to be approximately equivalent, the Region must provide their engineering analysis supporting the pavement type selection. The fact that these are not easily quantified does not lesson their importance; in fact these factors may be the overriding reason for making the final pavement type selection. These decision factors should be carefully reviewed and considered, by WSDOT engineers most knowledgeable of the corridor and the surrounding environment.

When offering the engineering analysis for pavement type selection, the Region must not use reasoning or examples that have already been taken into account within the pavement design analysis or the life cycle cost analysis. Examples of reasoning that should not be presented in the engineering analysis include:

1. Availability of funds for the more expensive pavement type.
2. Supporting the choice for pavement type based on ESALs or ADT (already accounted for in the life cycle cost analysis).
3. Supporting the choice for pavement type based on user delay (already accounted for in the life cycle cost analysis).

The Region should include the engineering reasons that drive the selection of one pavement type over another, given that their life cycle costs are approximately equivalent. Additional considerations, though not inclusive or exclusive, are found in APPENDIX 4. Not all factors will come into play on every project, nor will all factors have equal weight or importance on each project. Many of the factors are synergistic, combining or subtracting, depending on the selection and many of the factors are interrelated. Staff intimately familiar with the design goals of the entire project, or entire corridor, should make the engineering analysis evaluations.

4.5 SUBMITTAL PROCESS

The pavement type selection, including all applicable subsections (pavement design analysis, cost estimate and life cycle cost analysis, and engineering analysis) shall be submitted electronically to the Pavement Design Engineer at the HQ Materials Laboratory. The pavement type selection analysis shall be reviewed and distributed to the Pavement Type Selection Committee (APPENDIX 5) for approval. The report submittal shall include detailed explanation of the various applicable items, as those outlined above, that supports the selection of the recommended pavement type.

5. NEW FLEXIBLE PAVEMENT DESIGN

5.1 DESIGN PROCEDURES

"New flexible pavement design" shall include reconstructed as well as all new pavement structures.

The basic design procedure for flexible pavement structures will be according to the *AASHTO Guide for Design of Pavement Structures (1993)* and information contained in this guide. Further, certain minimum layer thicknesses as well as maximum lift thicknesses are controlled by requirements contained within WSDOT's Standard Specifications for Road, Bridge, and Municipal Construction (which also describes other pavement material requirements such as grading, fracture, cleanliness, etc.).

5.2 DETERMINATION OF PAVEMENT LAYER THICKNESSES

5.2.1 INTRODUCTION

Layer thicknesses and total pavement structure over subgrade soils for flexible pavements are fundamentally based on four criteria:

- Depth to provide a minimum level of serviceability for the design period,
- Depth to prevent excessive rutting,
- Depth to prevent premature fatigue cracking of the HMA layers, and
- Depth to provide adequate frost depth protection.

5.2.2 MAINLINE ROADWAY

The structural design of mainline flexible pavements can be broadly divided into those with fewer than 500,000 ESALs for the design period and those greater than 500,000 ESALs. Those pavements with fewer than 50,000 ESALs/year and ADT less than 2,000 are classified as low volume roadways and shall be considered for a bituminous surface treatment (Class A). For pavements with higher ESAL and ADT levels, an HMA surfacing shall be considered. Table 10 provides typical layer thicknesses for HMA surfaced flexible pavements for ESAL levels greater than 500,000. Figure 7 shows options for flexible pavement structures. Structural designs other than those shown in Table 10 can be used if justified by use of job specific input values in the *AASHTO Guide for Design of Pavement Structures (1993)*. (Note: the input values used to prepare Table 10 are shown at the bottom of the table. Other input values used by Region personnel must be approved by the HQ Materials Laboratory – Pavements Division).

Table 11 and Table 12 provide overviews of typical layer thicknesses for flexible pavements with design ESAL levels of 500,000 or less. The bituminous surface treatment (BST) surface course is considered the most economical choice for low ESAL pavements.

The reliability levels used in Table 10, Table 11, and Table 12 correspond to:

- National Highway System (NHS)
  - NHS Trunk (Interstate): 95%
  - NHS Branch (Principal Arterial): 85%
- Non Federal Aid (Minor Arterial, Collector): 75%

5.2.3 RAMPS, FRONTAGE ROADS, AND WEIGH STATIONS

Ramps shall be designed for the expected traffic.

Frontage roads and weigh stations that are maintained by WSDOT shall be designed in accordance with the *AASHTO Guide for Design of Pavement Structures (1993)*. Frontage roads that counties and cities are to accept and maintain but constructed by WSDOT shall be designed to the standards of the accepting agency.

The total depth of the pavement section must exceed one-half of the maximum expected depth of freezing when the subgrade is classified as a frost susceptible soil.

5.2.4 REST AREAS

The minimum flexible pavement requirements for rest area roadways and parking areas are:

• Access Roads and Truck Parking	0.50 ft HMA 0.30 ft CSBC
• Car Parking	0.35 ft HMA 0.30 ft CSBC

Project specific traffic and subgrade soil conditions may require thicker pavement layers. Such designs shall be done in accordance with the *AASHTO Guide for Design of Pavement Structures (1993)*. The total depth of the pavement section must exceed one-half of the maximum expected depth of freezing when the subgrade is classified as a frost susceptible soil.

5.2.5 SHOULDERS

The minimum requirements for flexible pavement shoulders are:

• Interstate	0.35 ft HMA 0.35 ft CSBC Variable Depth Gravel Base*
• Non-Interstate	0.25 ft HMA 0.35 ft CSBC Variable Depth Gravel Base*

* The Gravel Base shall extend to the bottom of the mainline base course.

Project specific traffic and subgrade soil conditions may require thicker pavement layers. Such designs shall be done in accordance with the *AASHTO Guide for Design of Pavement Structures (1993)*.

The total depth of the pavement section must exceed one-half of the maximum expected depth of freezing when the subgrade is classified as a frost susceptible soil.

Table 10. Flexible Pavement Layer Thicknesses for New or Reconstructed Pavements

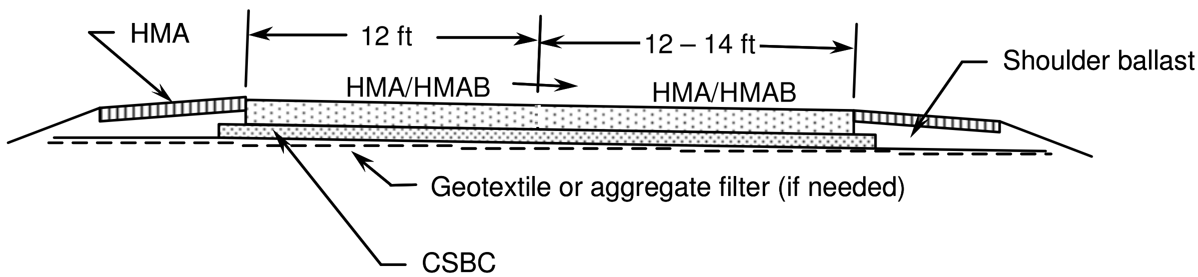
Design Period ESALs	Subgrade Condition	Layer Thicknesses,[1] ft								
		Reliability = 75%			Reliability = 85%			Reliability = 95%		
		HMA	HMAB	CSBC[2]	HMA	HMAB	CSBC[2]	HMA	HMAB	CSBC[2]
500,000-1,000,000	Poor	0.35	—	1.25	0.40	—	1.30	0.45	—	1.45
	Average	0.35	—	0.65	0.40	—	0.70	0.45	—	0.75
	Good	0.35	—	0.25	0.40	—	0.25	0.45	—	0.25
1,000,000-5,000,000	Poor	0.35	0.50	0.30	0.35	0.55	0.30	0.35	0.65	0.30
	Average	0.35	0.30	0.30	0.35	0.35	0.30	0.35	0.45	0.30
	Good	0.25	0.25	0.30	0.25	0.25	0.30	0.35	0.25	0.30
5,000,000-10,000,000	Poor	0.35	0.60	0.35	0.35	0.65	0.35	0.35	0.75	0.35
	Average	0.35	0.40	0.35	0.35	0.45	0.35	0.35	0.50	0.35
	Good	0.25	0.30	0.35	0.35	0.25	0.35	0.35	0.30	0.35
10,000,000-25,000,000	Poor	0.35	0.70	0.45	0.35	0.75	0.45	0.35	0.90	0.45
	Average	0.35	0.45	0.45	0.35	0.50	0.45	0.35	0.60	0.45
	Good	0.35	0.25	0.45	0.35	0.30	0.45	0.35	0.40	0.45
25,000,000-50,000,000	Poor	0.35	0.80	0.45	0.35	0.90	0.45	0.35	1.00	0.45
	Average	0.35	0.55	0.45	0.35	0.60	0.45	0.35	0.75	0.45
	Good	0.35	0.35	0.45	0.35	0.40	0.45	0.35	0.50	0.45
50,000,000-75,000,000	Poor	0.35	0.90	0.45	0.35	1.00	0.45	0.35	1.05	0.45
	Average	0.35	0.60	0.45	0.35	0.70	0.45	0.35	0.80	0.45
	Good	0.35	0.40	0.45	0.35	0.45	0.45	0.35	0.55	0.45

[1] *AASHTO Guide for Design of Pavement Structures (1993)* for flexible pavements and the following inputs:

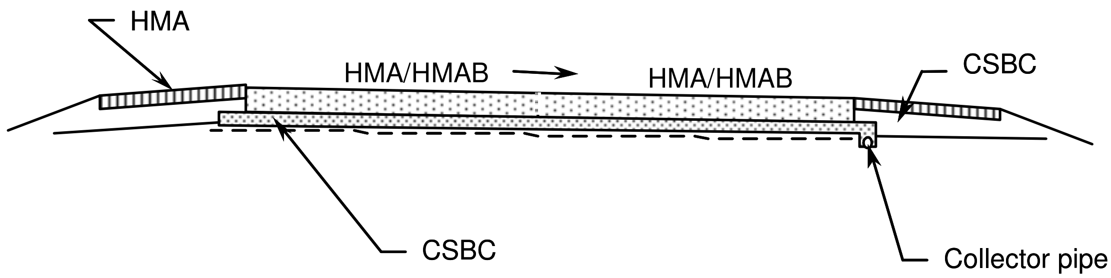
- $\Delta PSI = 1.5$
- $S_0 = 0.50$
- $m = 1.0$
- $a_{HMA} = 0.44$
- $a_{HMAB} = 0.44$
- $a_{CSBC} = 0.13$
- Subgrade Condition (effective modulus)
  - Poor: $M_R = 5{,}000$ psi
  - Average: $M_R = 10{,}000$ psi
  - Good: $M_R = 20{,}000$ psi

  (Note: Effective modulus represents the subgrade modulus adjusted for seasonal variation)

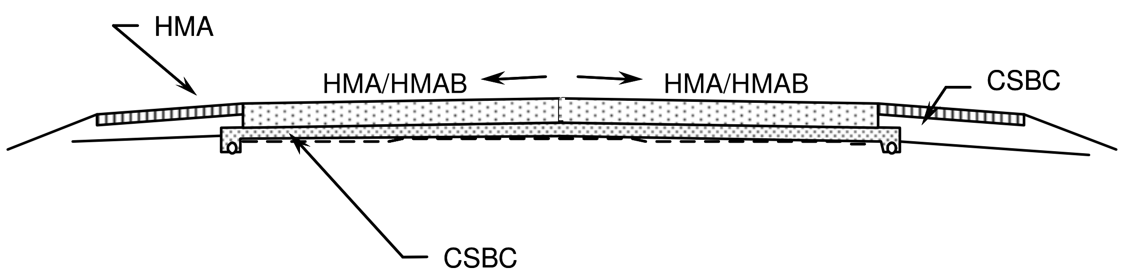
[2] GB may be substituted for a portion of CSBC when the required thickness of CSBC 0.70 ft. The minimum thickness of CSBC is 0.35 ft when such a substitution is made.



(a) Base Drains to Daylighted Shoulder Ballast



(b) Base Drains to Longitudinal Edge Drain



(c) Base Drains to Longitudinal Edge Drains – Crown Section

Figure 7. Flexible Pavement Sections

Table 11. Flexible Pavement Layer Thicknesses for Low ESAL Levels and New or Reconstructed Pavements—BST Surfaced

Design Period ESALs	Subgrade Condition	Required SN	Layer Thicknesses,[1] ft Reliability = 75%	
			BST[3] (Class A)	CSBC[2]
< 100,000	Poor	2.53	0.08	1.50
	Average	1.93	0.08	1.10
	Good	1.45	0.08	0.90[4]
100,000-250,000	Poor	2.95	0.08	1.80
	Average	2.25	0.08	1.30
	Good	1.71	0.08	1.00
250,000-500,000	Poor	3.31	0.08	2.00
	Average	2.53	0.08	1.50
	Good	1.93	0.08	1.10

[1] *AASHTO Guide for Design of Pavement Structures (1993)* for flexible pavements and the following inputs:

- $\Delta PSI = 1.7$
- $S_0 = 0.50$
- $m = 1.0$
- $a_{BST} = 0.20$ (assumes $E_{BST} = 100{,}000$ psi)
- $a_{CSBC} = 0.13$
- $SN = a_{BST}$ (1") + 0.13 (CSBC)
- Subgrade Condition (effective modulus)
  - Poor: $M_R = 5{,}000$ psi
  - Average: $M_R = 10{,}000$ psi
  - Good: $M_R = 20{,}000$ psi

  (Note: Effective modulus represents the subgrade modulus adjusted for seasonal variation)

[2] GB may be substituted for a portion of CSBC when the required thickness of CSBC 0.70 ft. The minimum thickness of CSBC is 0.35 ft when such a substitution is made.

[3] BST Class A assumed thickness = 0.08 ft

[4] CSBC thickness increased for a total pavement structure of approximately 1.00 ft based on moisture and frost conditions.

Table 12. Flexible Pavement Layer Thicknesses for Low ESAL Levels and New or Reconstructed Pavements—HMA Surfaced

Design Period ESALs	Subgrade Condition	Layer Thicknesses,[1] ft Reliability = 75%	
		HMA	CSBC[2]
< 100,000	Poor	0.25	0.80
	Average	0.25	0.75[3]
	Good	0.25	0.75[3]
100,000-250,000	Poor	0.30	0.95
	Average	0.30	0.70[3]
	Good	0.30	0.70[3]
250,000-500,000	Poor	0.35	1.00
	Average	0.35	0.65[3]
	Good	0.35	0.65[3]

[1] *AASHTO Guide for Design of Pavement Structures (1993)* for flexible pavements and the following inputs:

- ΔPSI = 1.7
- S_0 = 0.50
- m = 1.0
- a_{HMA} = 0.44
- a_{CSBC} = 0.13
- Subgrade Condition (effective modulus)
  - Poor: M_R = 5,000 psi
  - Average: M_R = 10,000 psi
  - Good: M_R = 20,000 psi

  (Note: Effective modulus represents the subgrade modulus adjusted for seasonal variation)

[2] GB may be substituted for a portion of CSBC when the required thickness of CSBC 0.80 ft. The minimum thickness of CSBC is 0.35 ft when such a substitution is made.

[3] CSBC thickness increased for a total pavement structure of 1.00 ft based on moisture and frost conditions.

6. NEW RIGID PAVEMENT DESIGN

6.1 DESIGN PROCEDURES

"New rigid pavement design" shall include reconstructed as well as all new pavement structures.

The basic design procedure for rigid pavement structures will be that as set forth in the *AASHTO Guide for Design of Pavement Structures (1993)* and contained in this guide. Further, certain minimum layer thicknesses are controlled by requirements contained within WSDOT's *Standard Specifications for Road, Bridge and Municipal Construction* (which also describes other pavement material requirements such as grading, fracture, cleanliness, etc.).

The principal type of rigid pavement used by WSDOT in the past and which will be continued for the foreseeable future is a plain, jointed PCC pavement (with or without dowel bars).

6.2 DETERMINATION OF PAVEMENT LAYER THICKNESSES

6.2.1 INTRODUCTION

Based on the past performance of PCC pavement on the state route system under a variety of traffic conditions (various ESAL levels) and on city streets (such as the city of Seattle), it is advisable to use slab thicknesses of 0.65 feet or greater even if the ESAL levels would suggest that lesser slab thicknesses would be adequate. A slab thickness of 0.65 feet or greater provides some assurance of adequate long-term performance given that other design details are adequately accommodated. Past PCC pavement performance also suggests that rigid pavement can be designed for an initial performance period of 50 years. This is of significant benefit where rehabilitation and maintenance activities are highly constrained (such as urban roads, streets, and interstate pavements).

In the past, base depths under rigid pavements were determined primarily by the requirement for support of construction traffic. Currently, it is recognized that the base course directly beneath PCC slabs is a critical element in the performance of PCC pavement. Previous to this Guide, asphalt treated base (ATB) was used to support construction traffic prior to placement of PCC pavement. Recent WSDOT experience indicates degradation of the ATB material beneath various Interstate PCC pavements. For this reason, HMAB is recommended as the supporting layer for PCC slabs.

Where HMAB is to be used beneath PCC and the HMAB is placed on the grading contract, a minimum of 0.10 foot of the layer should be reserved for placement on the PCC paving contract for the necessary leveling operations. For this lift of material, special provisions should provide for a maximum aggregate size appropriate for the lift depth. Allow sufficient extra material to permit preleveling and making up for deficiency and settlement of existing grade. HMAB is most effective in waterproofing the grade when the grade is in reasonably good shape. The benefits of a waterproofing treatment under the ultimate pavement is largely lost if an untreated base is placed directly over the subgrade and then allowed to stand over the winter without an HMAB "surface".

6.2.2 MAINLINE ROADWAYS

Table 13 and Table 14 provide typical PCC slab thicknesses for various levels of ESALs and reliability. The input values used to produce the tables are shown beneath each table. The slab thicknesses were calculated using a J factor of 3.4 (Table 13) or J = 2.7 (Table 14), which are estimates of contraction joint performance. A J factor of 3.4 is considered a minimal (or limited) performance standard for PCC pavement contraction joints. A J factor of 2.7 represents improved or enhanced PCC pavement contraction joint performance.

Table 13. PCC Slab Thicknesses for Limited Contraction Joint Performance for New or Reconstructed Pavements (Non-Doweled Joints and CSBC)

Design Period ESALs	Slab Thickness,[1] ft		
	Reliability 75%	Reliability 85%	Reliability 95%
<5,000,000	0.75	0.80	0.85
5,000,000-10,000,000	0.80	0.90	0.95
10,000,000-15,000,000	0.90	0.95	1.00

[1] *AASHTO Guide for Design of Pavement Structures (1993)* for plain jointed pavement and the following inputs:

- J = 3.4
- E_c = 4,000,000 psi
- ΔPSI = 1.5
- S_c' = 650 psi
- S_0 = 0.40
- C_d = 1.0
- k = 200 (assumes use of CSBC)

Thicknesses (and associated k value) assume firm and unyielding subgrade conditions.

Table 14. PCC Slab Thicknesses for Limited Contraction Joint Performance for New or Reconstructed Pavements (Non-Doweled Joints and HMAB)

Design Period ESALs	Slab Thickness,[1] ft		
	Reliability 75%	Reliability 85%	Reliability 95%
<5,000,000	0.70	0.75	0.85
5,000,000-10,000,000	0.80	0.85	0.95
10,000,000-25,000,000	0.95	1.00	1.10

[1] *AASHTO Guide for Design of Pavement Structures (1993)* for plain jointed pavement and the following inputs:

- J = 3.4
- E_c = 4,000,000 psi
- ΔPSI = 1.5
- Sc' = 650 psi
- S_0 = 0.40
- C_d = 1.0
- k = 400 pci (assumes use of HMAB)

Thicknesses (and associated k value) assume firm and unyielding subgrade conditions.

Table 15. PCC Slab Thicknesses for Improved Contraction Joint Performance for New or Reconstructed Pavements (Doweled Joints and CSBC)

Design Period ESALs	Slab Thickness,[1] ft		
	Reliability 75%	Reliability 85%	Reliability 95%
<25,000,000	0.85	0.90	1.00
25,000,000-50,000,000	0.95	1.00	1.10
>50,000,000	1.00	1.10	1.20

[1] *AASHTO Guide for Design of Pavement Structures (1993)* for doweled, plain jointed pavement and the following inputs:

- J = 2.7
- E_c = 4,000,000 psi
- ΔPSI = 1.5
- Sc' = 650 psi
- S_0 = 0.40
- C_d = 1.0
- k = 200 pci (assumes use of CSBC)

Thicknesses (and associated k value) assume firm and unyielding subgrade conditions.

Table 16. PCC Slab Thicknesses for Improved Contraction Joint Performance for New or Reconstructed Pavements (Doweled Joints and HMAB)

Design Period ESALs	Slab Thickness,[1] ft		
	Reliability 75%	Reliability 85%	Reliability 95%
<25,000,000	0.75	0.80	0.90
25,000,000-50,000,000	0.85	0.90	1.00
>50,000,000	0.90	0.95	1.05

[1] *AASHTO Guide for Design of Pavement Structures (1993)* for doweled, plain jointed pavement and the following inputs:

- J = 2.7
- E_c = 4,000,000 psi
- ΔPSI = 1.5
- Sc' = 650 psi
- S_0 = 0.40
- C_d = 1.00
- k = 400 pci (assumes use of HMAB)

Thicknesses (and associated k value) assume firm and unyielding subgrade conditions.

To achieve a J factor of 3.4, undoweled PCC slabs are placed on an HMAB with a free draining shoulder section. This section is shown in Figure 8(a).

To achieve improved contraction joint performance, dowel bars must be used at all contraction joints. For typical WSDOT PCC pavements, a J factor of 2.7 was used to develop the slab thicknesses shown in Table 15 and Table 16. This assumes that the doweled PCC pavement is placed as shown in Figure 8(b) and/or Figure 9. In addition, urban rigid pavement will have tied PCC shoulders, as shown in Figure 9.

PCC slab thicknesses other than those shown in Table 13 through Table 16 can be used if justified by job specific input values into the *AASHTO Guide for Design of Pavement Structures*

(1993). Such input values must be approved by the HQ Materials Laboratory – Pavements Division. For projects that are projected to have substantial numbers of transit vehicles (buses), doweled contraction joints shall be considered.

The reliability levels to use in Table 13 through Table 16 are the same as shown in Paragraph 5.2.2.

The total depth of the pavement section must exceed one-half of the maximum expected depth of freezing when the subgrade is classified as a frost susceptible soil.

6.2.3 RAMPS, FRONTAGE ROADS, AND WEIGH STATIONS

The same requirements apply to rigid pavement ramps and frontage roads as for flexible pavements as noted in Paragraph 5.2.3.

6.2.4 REST AREAS

The minimum rigid pavement requirements for rest area roadways and parking areas are:

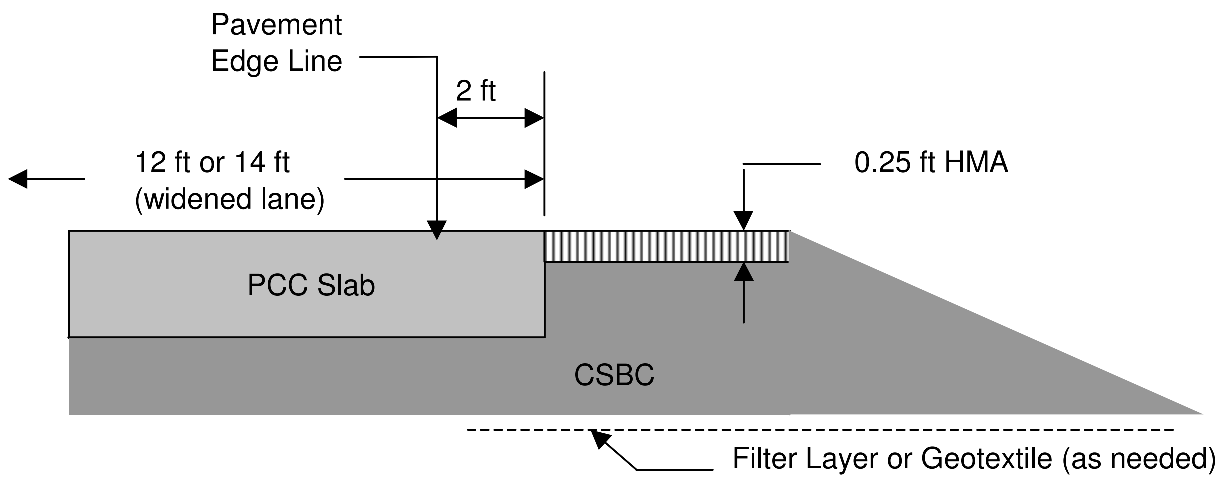
• Access Roads and Truck Parking	0.75 ft PCC 0.30 ft HMAB
• Car Parking	0.65 ft PCC 0.30 ft HMAB

Project specific traffic and subgrade soil conditions may require thicker pavement layers. Such designs shall be done in accordance with the *AASHTO Guide for Design of Pavement Structures (1993)*.

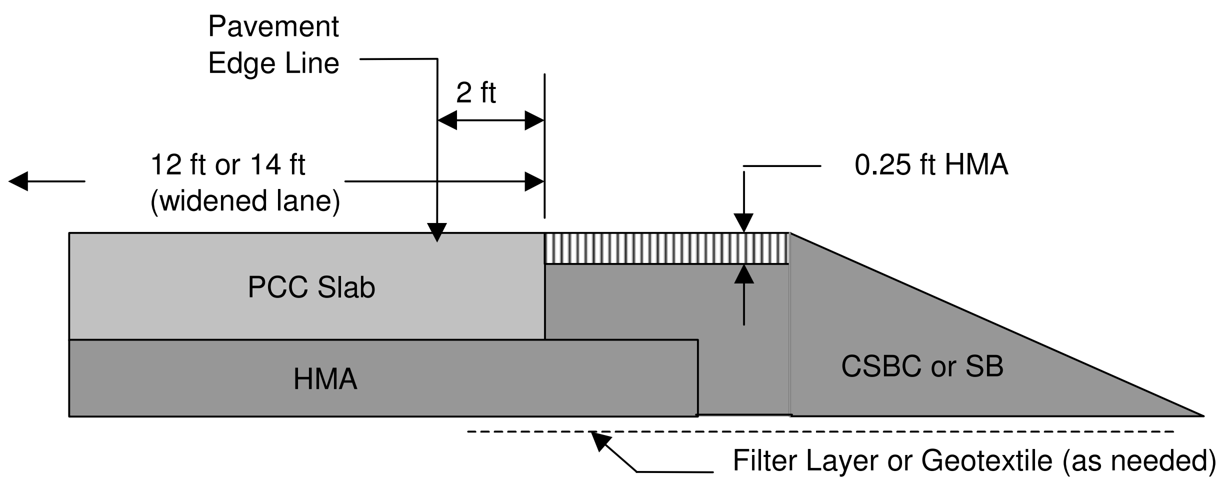
The total depth of the pavement section must exceed one-half of the maximum expected depth of freeze when the subgrade is classified as a frost susceptible soil.

6.2.5 SHOULDERS

When PCC shoulders are used they shall have the same PCC slab and base thicknesses as the mainline roadway PCC (refer to Figure 9). Additionally, the shoulder and mainline roadway PCC shall be tied together with deformed steel bars. See also paragraph 7.1.5.3.4.



(a) Undoweled PCC Slabs on CSBC



(b) Doweled PCC Slabs on HMAB

Figure 8. Typical PCC Pavement Sections

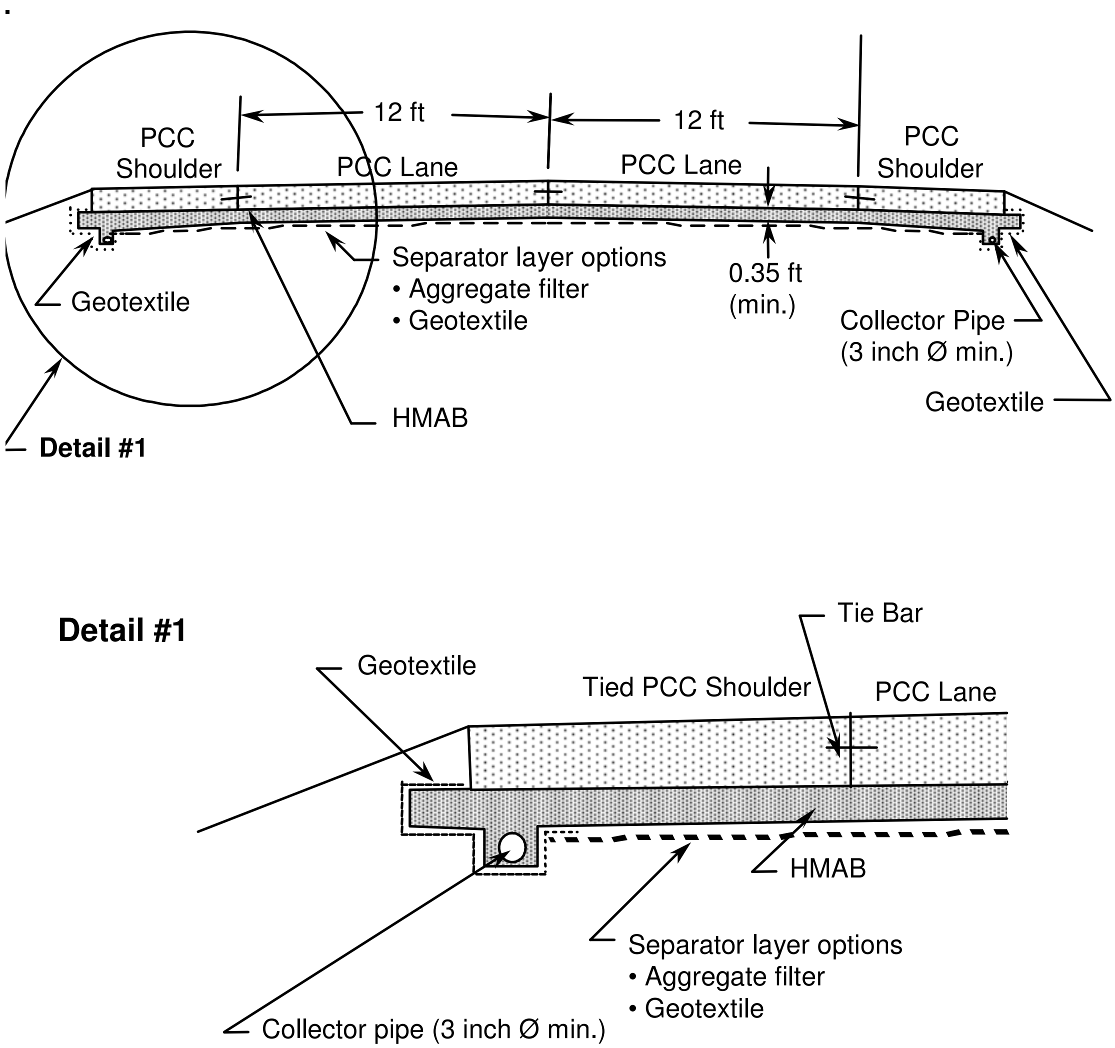


Figure 9. PCC Pavement with Tied Concrete Shoulder

7. PAVEMENT REHABILITATION

7.1 REHABILITATION DESIGN PROCESS

7.1.1 BASIC ELEMENTS

A pavement rehabilitation project usually deals with existing pavements that show obvious signs of distress or failure. As a result, the amount of data acquired for each survey may vary in both quantity and detail, depending on the condition of the roadway and the amount of data on file, such as earlier soils reports. The survey data is incorporated into the soil report where the scope of the project warrants a soil survey (that is, significant widening, cuts or fills). The survey includes pavement deflection data, descriptions and photographs of typical pavement conditions, road life history, pavement cores, base course and subgrade samples, and a review of drainage features. The survey shall be oriented toward analyzing the existing roadway conditions so that a reasonable definition of the special problems and structural needs of the roadway may be made.

7.1.2 DEFLECTION SURVEY

A pavement deflection survey is performed on selected projects by the HQ Materials Laboratory – Pavements Division. This survey shall be conducted before the rehabilitation report to aid the Regional Materials Engineer in coring and sampling of each project. The deflection survey shall be conducted, when possible, either in late fall or early spring. The Regional Materials Engineer shall coordinate with the HQ Materials Laboratory – Pavements Division so that most of the deflection surveys are conducted during one time period each year. After conducting the deflection surveys, the HQ Materials Laboratory – Pavements Division will report the results of the survey to the Regional Materials Engineer.

7.1.3 DESIGN PERIOD

The rehabilitation design period is the time from rehabilitation construction to a terminal condition. Rehabilitation designs for an HMA overlay will typically have a design period of 15 years, six to eight years for a BST, and ten to 15 years for dowel bar retrofit. This design period can be changed, if needed, when justified by project specific conditions.

7.1.4 TRAFFIC DATA

Traffic data from the TRIP's traffic file will be used on most projects, as contained in the WSPMS. Where the HQ Materials Laboratory – Pavements Division or Regional Materials Engineer believes the data in the file is not adequate, a special traffic count on the project can be requested to verify the data. If the region does not have personnel to conduct the traffic counts, the Transportation Data Office shall be contacted for assistance.

For Superpave mix designs, the design ESAL calculation will be based on 15 years.

7.1.5 OTHER PAVEMENT ISSUES

This section describes the various details involved with new pavement construction and rehabilitation.

7.1.5.1 General

7.1.5.1.1 Mainline Shoulders

Mainline shoulders shall be designed to the equivalent HMA or PCC and base depths as the mainline roadway, especially in areas where the shoulder could potentially become a traveled lane. Consideration for a lesser depth will be given in rural areas or where shoulders will not become a future traveled lane.

7.1.5.1.2. Pavement Widening

Roadway widening to accommodate HOV, truck climbing lane, or capacity improvements shall be constructed with like pavement of the mainline. Widening in kind will minimize potential surface irregularities between adjacent lanes and minimize future rehabilitation challenges. Deviations from widening with like pavement type shall be determined using the WSDOT Pavement Type Selection process (as outlined in Section 4 of this document).

7.1.5.2 Flexible Pavements

7.1.5.2.1 Prelevel

The use of prelevel prior to placement of an overlay is strictly limited to the correction of safety related deficiencies unless otherwise stated in the Rehabilitation Report. Safety-related uses of prelevel are as follows:

1. To remove hazardous “spot” locations, e.g., ponding areas or to restore proper pavement drainage at a specific location.
2. To correct deficient superelevation or cross slope when the deficiency is the cause of operational problems as determined from an accident history analysis.
3. To address pavement rutting specifically identified in the Rehabilitation Report (rutting of < ” will generally be addressed with the overlay).

When prelevel is warranted as outlined above, it must be clearly documented in the design recommendations and carefully detailed in the contract PS&E so that the use is clearly apparent to the contractor and the construction project engineer.

7.1.5.2.2 Crack Sealing

The item “crack sealing” will only be used when specifically recommended in the Rehabilitation Report. Crack sealing will be done only on cracks ¼ inch and wider, see Standard Specification 5-04.3(5)C. Minor cracks will be addressed by the use of tack coat.

7.1.5.2.3. Pavement Repair

As WSDOT HMA pavements become thicker, due to successive overlays, failures tend to be limited to the wearing surface. Distress seen in thicker HMA pavements (generally greater than seven inches) typically occurs as top down cracking. Top down cracks often penetrate only the wearing surface of a roadway and do not affect the aggregate base or subgrade. Options for rehabilitating pavements with top down cracking include rotomilling and inlaying or overlaying depending upon the extent and depth of the distress. In most cases pavement coring will easily identify the depth of the required pavement repair.

Thinner pavements (generally less than six inches) often experience distress throughout the HMA thickness and sometimes into the aggregate base and subgrade. In these cases, full depth replacement of the HMA may be warranted, however, the repair of the pavement failures can range from removing the entire pavement section to only the depth of the last overlay. Coring shall be performed to isolate the depth of required repair. Depending on the distress, removal and replacement of aggregate base and subgrade may be necessary.

It is important that the Project Offices work closely with the Region Materials Office to determine the cause and extent of the pavement failures. The Region Materials Office will determine if coring is necessary to evaluate the pavement condition.

While pavement repair is preferred to totally remove the distressed pavement, increasing the overlay depth in localized areas can also be considered if conditions warrant. The additional cost of the overlay, however, shall be compared to the cost of providing pavement repair.

7.1.5.2.4. Fog Sealing

Fog sealing open-graded pavements, such as Modified Class D, are recommended one year after placement and every four years thereafter. This maintenance activity will minimize pavement ravelling, extend pavement life, and provide the lowest life cycle cost. While the one and four year fog seals are recommended, each region shall determine fog sealing recommendations based on actual open-graded roadway performance. Shoulders shall be fog sealed based on the Region Materials Office recommendations.

7.1.5.2.5. Recessed Lane Markers and Rumble Strips in HMA Roadways

Recessed lane markers and methyl methacrylate striping, thermoplastic stop bares, arrows, or other coated materials shall be removed prior to placement of the HMA overlay.

Rumble strips may be overlaid with a minimum depth of 0.15 feet HMA as long as there is no shift in the existing lane configuration that will cause the wheel path to cross over the underlying rumble strips. If this is the case, reflection cracking of the underlying rumble strip may occur.

On HMA inlay projects where the rumble strips need replacement, the width of the inlay can be increased outside of the fogline to include the rumble strip area.

Rumble strips on shoulders that will carry traffic as a detour shall be preleveled or ground and inlayed with a minimum depth of 0.15 feet HMA. A typically rehabilitation option is to rotomill and inlay a three foot width from the fogline towards the shoulder edge.

Rumble strips located between directional traffic shall be preleveled or rotomilled and inlayed.

7.1.5.2.6. Recessed Lane Markers and Rumble Strips in BST Roadways

Placement of recessed lane markers and rumble strips is discouraged in BST roadways. However, if either is used the existing BST surfacing should be at least 0.25 ft thick. For subsequent BST overlays, the recessed lane marker or rumble strip area should be preleveled with HMA prior to BST placement.

7.1.5.2.7. Use of Open Graded Emulsion Asphalt Pavement (OGEAP)

OGEAP, when placed over existing HMA or BST, has shown early performance problems. The typical distress has been ravelling of the OGEAP layer. Placing a chip seal or HMA overlay does not prevent performance problems and can accelerate stripping. Therefore, the use of OGEAP as an overlay of an existing HMA or BST pavement is not recommended. However, OGEAP placed on a free draining base has not shown severe performance problems.

7.1.5.2.8. BST over New HMA Overlays

BST placed over newly placed HMA is generally not warranted. Where BST is placed for friction purposes, the need shall be clearly substantiated by the region Materials Engineer with supporting friction data. For routes reverting back to BST after an HMA overlay, BST shall be placed when the HMA overlay is due for rehabilitation as determined by the WSPMS.

7.1.5.2.9. HMA Paving Depth

The Roadway Paving Program cost estimate is based on a pavement overlay depth of 0.15'. The required depth for an HMA overlay shall be as recommended in the Rehabilitation Report. Every effort should be made to keep structural overlays to the 0.15' depth; however, in some cases this may not be possible due to existing structural conditions. Pavement designs greater than 0.15' require a detailed analysis, including a mechanistic-empirical pavement design, justifying the increase in overlay thickness.

Pavement thicknesses shall not be arbitrarily increased based on perceived concerns that the underlying layers will delaminate on a rotomill and inlay project. A thicker lift can be approved, however, cores must substantiate that a delaminated layer exists.

7.1.5.2.10. Tack Coat

A tack coat is required between all HMA layers (new construction and overlay).

7.1.5.2.11. Correcting Shoulder Slopes

Roadways with a 0.02 ft/ft cross slope on the lanes and 0.05 ft/ft on the shoulders may be corrected provided the shoulder width is four feet or less. On roadways with shoulders wider than four feet, the correction will be deferred depending on funding.

7.1.5.2.12. Removal of Open Graded Pavements Prior to Overlays

Open-graded pavements such as Class D, Modified D, or OGEAP shall be removed prior to overlaying with dense-graded or open-graded asphalt (including shoulders). Removal of the open-graded asphalt layer is necessary to avoid stripping of the open-graded layer. On lower volume roadways, cold in-place recycling of an OGEAP layer is also an acceptable rehabilitation alternative.

On rotomill and inlay projects, where only the travelled lanes are rehabilitated, open-graded pavements may remain on the shoulders for many rehabilitation cycles. However, where there is potential for the existing shoulder to become a travelled lane, the open-graded asphalt layer shall be removed prior to any future overlays.

7.1.5.2.13. BST to HMA Conversion Process

Pavements with less than 50,000 directional ESALs per year **and** AADT less than 2,000 are designated as bituminous surface treatments. Exceptions (such as paving through small cities, limited BST use, etc.) to this policy are evaluated on a case-by-case basis. If the Region desires a change in the designated pavement type, the following procedure shall be followed:

1. Regions shall submit all requests for change in pavement type designation to the State Pavement Engineer.
   a. Submittal will require reasoning for proposed change.
   b. Submittal will require a pavement status report (statement of due and past due miles).
   c. Pavement must be due no later than the requested biennium for conversion and must be requested no later than two years (in June) prior to the construction biennium. For example, a pavement due in the 05-07 biennium, the State Pavement Engineer must receive the conversion request by May 2003.
2. State Pavement Engineer will inform the HQ Strategic Planning and Programming Office if approved.

If the HQ Strategic Planning and Programming denies the request, they will inform the Region and the State Pavement Engineer that request has been denied and the reasoning for this denial.

7.1.5.3 PCC

7.1.5.3.1. Intersections Limits

The limits for reconstruction with PCC shall be determined based on an evaluation of the existing pavement conditions. The area of pavement rutting or distress shall be limited to the vehicle start and stop areas. The major arterial approach legs to intersections may require PCC from 200 to 500 feet back from the crosswalk.

7.1.5.3.2. Dowel Bar Type (see Appendix 1 for additional details)

1. Western Washington
    a. Preferred Dowel Bar Type: Stainless steel alternates
    b. Preferred Dowel Bar Spacing:
        i. Truck Lanes: twelve dowel bars per joint
        ii. Non-truck lanes: eight dowel bars per joint (four per wheel path)

2. Mountain Passes (greater than 2,000 foot elevation)
    a. Preferred Dowel Bar Type: Stainless steel alternates
    b. Preferred Dowel Bar Spacing:
        i. Truck Lanes: twelve dowel bars per joint
        ii. Non-truck lanes: eight dowel bars per joint (four per wheel path)

3. Eastern Washington
    a. Preferred Dowel Bar Type: Non-stainless, corrosion resistant alternates
    b. Preferred Dowel Bar Spacing:
        i. Truck Lanes: twelve dowel bars per joint
        ii. Non-truck lanes: eight dowel bars per joint (four per wheel path)

(See also APPENDIX 1)

7.1.5.3.3. HMAB

HMAB used as base under PCC shall be designed in accordance with Standard Specification 9-03.8. Test requirements and mix criteria meeting the 3 - 10 million ESAL range, an aggregate gradation meeting either the ½ inch or ¾ inch, and a base grade asphalt binder (PG 58-22 for Western Washington, PG 58-34 for Mountain Passes, and PG 64-28 for Eastern Washington) will be appropriate.

7.1.5.3.4. Use of Widened Lane

In urban roadways, it is recommended that shoulders be constructed with PCC, tied and dowelled. If shoulders are constructed with HMA, at a minimum, the right most lane (truck lane) shall be constructed 14 feet wide and striped at 12 feet.

7.1.5.3.5. PCC Shoulders

Dowel bars may be omitted from the left shoulder if the shoulder is never expected to carry traffic loads and is only to experience breakdown traffic. For the right shoulder, two options are applicable:

1. Construct a 14 ft wide dowelled right lane, stripe at 12 ft and only tie the concrete shoulder.
2. Construct a 12 ft wide dowelled right lane with a tied and dowelled concrete shoulder.

Any shoulder that has a potential for being used as traveled way (shifting traffic lanes onto the shoulders to gain an extra lane, for example) should be evaluated for dowel bar placement. If the shoulder requires dowel bars based on the above review, then dowel bars placement and type must match the adjacent mainline selection.

Shoulders that have the possibility of being used as a traveled lane should be evaluated to determine the need for dowels.

8. PAVEMENT REHABILITATION REPORT

A pavement rehabilitation report is required for all HMA and PCC rehabilitation and new construction projects and is recommended for BST overlays where structural problems are evident. The Regional Materials Engineer will prepare the report for review by the HQ Materials Laboratory – Pavements Division, which will summarize the findings of the pavement rehabilitation survey, including discussion of special features or problems and recommendations concerning possible rehabilitative measures.

The report shall cover the following topics, insofar as they pertain to the project, and include any other information pertinent to the analysis of the pavement rehabilitation needs:

8.1 GENERAL

Description of the project using vicinity maps and plan views, status and scope of project, climatic conditions, traffic conditions (including 15 year design ESALs), possible construction contingencies, State Route number, milepost limits, project name, XL, OL or contract number, Project Item Number, funded biennium, and anticipated construction dates.

8.2 GEOLOGY

Pertinent topographic features as they relate to subgrade soil changes and pavement performance.

8.3 PAVEMENT CONDITION

Description and photographs of existing pavement conditions with reference to pavement distress, subgrade soils, geologic features, drainage, frost distress or traffic.

8.4 DRAINAGE AND WATER CONDITIONS

Description of pertinent drainage features such as ditches, subgrade drains, drainage blankets, etc., both functioning and non-functioning. Where wet subgrades are encountered, moisture contents should be determined.

8.5 PAVEMENT LAYER PROFILE

Provide pavement profile information showing layer depths (example shown in Figure 10) and limits as they relate to past contracts, along with core and subgrade soil data. When deflection data is available, core sampling shall be taken every 0.25 to 0.50 miles of the projects length. Special areas of distress, particularly frost distress, shall also be noted.

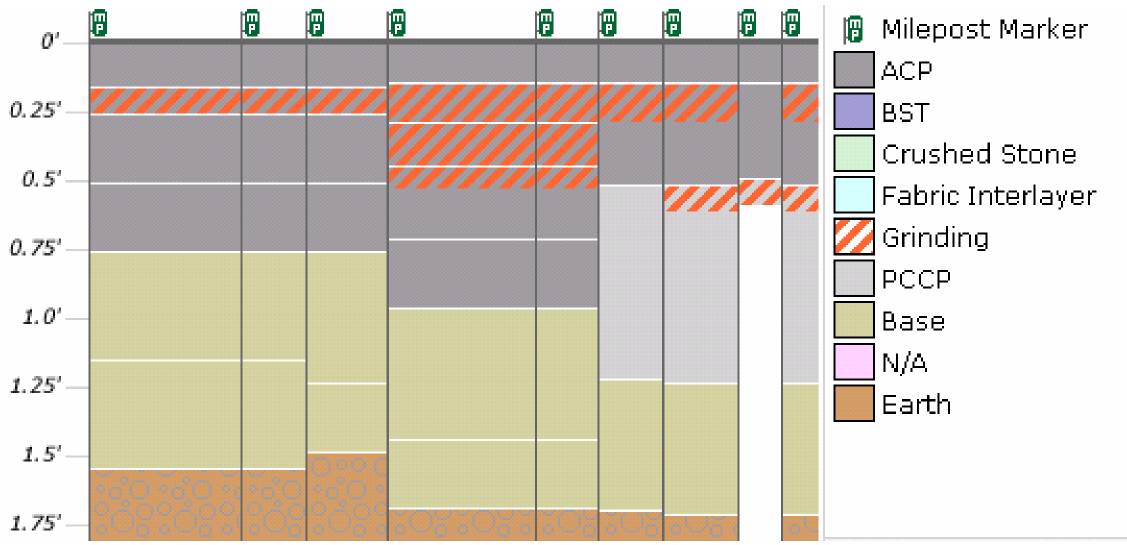


Figure 10. Illustration of Pavement Cross Section

8.6 MATERIALS

Source of materials to be used on the project along with special materials where warranted.

8.7 CONSTRUCTION CONSIDERATIONS

Items such as project timing, preleveling, digouts, subsealing, crack sealing, potential problems with materials sources, etc., should be covered.

8.8 SPECIAL FEATURES

Review any unique features pertinent to the project not covered under other topics.

8.9 RECOMMENDATIONS

Specific recommendations are warranted concerning pavement rehabilitation design, correction of special problems, unique use of materials or procedures, drainage features, frost distress corrections, etc.

8.10 HQ MATERIALS LABORATORY – PAVEMENT DIVISION REPORT

After the HQ Materials Laboratory – Pavement Division has evaluated the pavement rehabilitation report, tested base course and subgrade samples (where applicable), and verified pavement rehabilitation recommendations, a report, if necessary, is prepared reviewing and commenting on the various features and rehabilitation needs of the project. Otherwise, concurrence will be provided in a signature and date block provided on the Region Rehabilitation Report. This report is then returned to the Region Materials Engineer for distribution.

8.11 DESIGN PROCEDURES

8.11.1 OVERLAY DESIGN

8.11.1.1 HMA Overlays

HMA overlay design can be accomplished either by use of the mechanistic-empirical based scheme used in the Everpave©[6] computer program or the *AASHTO Guide for Design of Pavement Structures (1993)*, Part III, Chapter 5. The Everpave© program is for use with flexible pavements. The AASHTO procedure can be applied to either flexible or rigid pavement structures.

8.11.1.2 Granular Overlays (Cushion Courses)

The granular overlay system (often referred to as a "cushion course") is an alternative type of overlay for rehabilitating mostly low volume, rural roads (this does not necessarily imply a low number of ESALs). The overlay consists of a layer of densely compacted, crushed rock (CSBC) overlain by a generally thin surface layer.

8.11.1.3 PCC Overlays

Generally, only unbonded PCC overlays will be used if a PCC surfacing is selected. Bonded PCC overlays are not considered as a structural solution and have a higher than acceptable risk of premature failure. Unbonded PCC overlays will be designed by use of the *AASHTO Guide for Design of Pavement Structures (1993)*.

8.11.1.4 Dowel Bar Retrofit, Panel Replacement, and Diamond Grinding

Dowel bar retrofit, in conjunction with localized panel replacements (as necessary) and diamond grinding has proven to be a viable PCC pavement rehabilitation procedure in Washington State. This rehabilitation option restores transverse joint load transfer, repairs PCC panels that are distressed beyond repair, and provides a smooth riding pavement surface.

8.12 DETERMINATION OF PAVEMENT LAYER THICKNESSES

8.12.1 OVERLAYS

8.12.1.1 HMA Overlays — Structural

The minimum depth of HMA overlay required for structural applications will be 0.12 feet. Depths less than 0.12 feet are considered to be nonstructural overlays.

[6] Everpave© (Everseries Program) can be downloaded from - http://www.wsdot.wa.gov/biz/mats/Apps/EPG.htm

8.12.1.2 HMA Overlays — Nonstructural

A nonstructural overlay can be any depth that achieves adequate density during construction. For example, inch HMA can be successfully placed at depths of less than 0.08 foot in the proper paving conditions (including weather).

8.12.1.3 Minimum Lift Thickness

To ensure that adequate compaction can be obtained the following minimum lift thicknesses by class of mix shall be followed:

Class of Mix	**Minimum Lift Thickness (feet)**
inch HMA	0.08
½ inch HMA	0.12
¾ inch HMA	0.20
1 inch HMA	0.25

8.12.1.4 Granular Overlay

The surfacing depth can vary depending on local conditions and requirements; however, the CSBC depth shall not exceed 0.50 feet in order to achieve the maximum structural benefit.

8.12.1.5 PCC Overlay

The minimum, unbonded PCC slab thickness shall not be less than 0.65 ft unless a special analysis indicates otherwise. Normally, unbonded PCC overlay thicknesses will be about as thick as new PCC construction.

REFERENCES

1. Federal Highway Administration, “Techniques for Pavement Rehabilitation, Participant's Notebook”, FHWA-HI-90-022 (1987).

2. Dell'Isola, A. J. and S. J. Kirk, *Life Cycle Costing for Design Professionals*, McGraw-Hill, New York (1981).

3. Kleskovic, Peter Z., “A Discussion of Discount Rates for Economic Analysis of Pavements”, Draft Report, FHWA Pavement Division, February 1990.

4. Federal Highway Administration, “Life cycle Cost Analysis in Pavement Design”, FHWA-SA-98-079 (1998). http://199.79.179.19/OLPFiles/FHWA/010114.pdf

5. Pennsylvania Department of Transportation, “Publication 242 – Pavement Policy Manual”, April 6, 2001, ftp://ftp.dot.state.pa.us/public/Bureaus/BOMO/RM/Publication242.pdf.

6. Peterson, Dale E., NCHRP Synthesis of Highway Practice No. 122: Life cycle Cost Analysis of Pavements, Highway Research Board, National Research Council, Washington, D.C. (1985).

7. Office of Management and Budget (1992—updated 2003), “Guidelines and Discount Rates for Benefit-Cost Analysis of Federal Programs,” Circular No. A-94 (Revised), Original Publication October 29, 1992 but updated with new discount rates January 2003. http://www.whitehouse.gov/omb/circulars/a094/a094.html#ap-b

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1. NIST (1995), "Life Cycle Costing Manual for the Federal Energy Management Program," NIST Handbook 135, US Department of Commerce, 1995 Edition. http://www.energyworkshops.org/femp/2001/2001modules/handbk.pdf

APPENDIX 1 – DOWEL BAR TYPE SELECTION PROTOCOL

Dowel bars in Portland Cement Concrete (PCC) pavement have been proven to extend pavement life. Dowel bars transfer loads from panel to panel, supplementing the aggregate interlock at the panel joint. Aggregate interlock at the joint degrades over time, while dowel bars are expected to continue to be effective at load transfer for upwards of 50 years. WSDOT PCC pavements are designed to last 50 years, so it is critical that the dowel bars also survive, intact and functional, for this period.

Different materials used for dowel bars have different expected performance lives, given various exposures to weather and corrosive chemicals. Dowel bars in wet environments with exposure to salts/corrosive agents (either naturally from the environment, such as sea spray, or from chemical anti-icing compounds) are the harshest environments. Dowel bars placed in dry climates without exposure to salts/corrosive agents experience the mildest environment. For the same moisture and salt/corrosive environment, warmer climates would induce more corrosion than colder environments.

The purpose of this protocol is to balance risk and cost. In an unconstrained funding scenario, one would select the least risky dowel bar material: one most resistant to corrosion. Unfortunately, WSDOT will always be under some type of funding constraint. Risk and cost, for each type of dowel bar material, is illustrated in the following table:

Dowel Bar Type	Cost	Corrosion Resistance
Solid stainless steel	Most expensive	Best corrosion resistance
Stainless steel clad	⇓	⇓
Stainless steel sleeve with epoxy coated insert	⇓	⇓
MMFX-2 steel (patented steel using chromium below limits to qualify as stainless and microstructure to resist corrosion)	⇓	⇓
Epoxy coated	⇓	⇓
Black steel (uncoated)	Least expensive	Worst corrosion resistance

Corrosion resistance increases as does cost when moving from black steel to stainless steel dowels. Additionally, there is a direct link, then, between risk and cost: less risk, higher cost; lowest cost, greatest risk of corrosion before 50 years.

CLIMATE REGIMES

Wet climates promote corrosion in steel more than drier climates. In general, western Washington has the greatest exposure to moisture in PCC pavements. Most of eastern Washington is considerably drier, experiencing more snow but less rainfall, and less overall moisture, than western Washington.

CORROSION REGIMES

PCC pavement directly adjacent to salt water has a high-risk exposure to corrosive salts. Fortunately, little PCC pavement has this type of exposure in Washington State. The greatest exposure to corrosive salts will be in locations where the highway is regularly treated with salts/corrosive agents during the winter months. Mountain passes, particularly those with "clear pavement" protocols (wherein Maintenance maintains the highway in a snow/ice free condition) will have the greatest exposure.

TRAFFIC LOADING

Trucks present the greatest loading risk for load transfer between adjacent PCC pavement panels. Truck lanes (usually Lane 1 or Lane 2, depending on the total number of lanes and following WSDOT lane counting protocol which starts from the slow lane as lane 1 and works toward the fast lane) will have the greatest number of ESALs (Equivalent Single Axle Load, a measure of the traffic loading experienced by a pavement). Risk of load transfer failure increases with increasing ESALs. Lanes with the greatest truck traffic will need more dowels to ensure efficient load transfer. On multi-lane highways, the "auto" lanes (Lanes 3, 4 or 5) will typically have much fewer trucks. These lanes can probably be designed with fewer dowel bars per lane and still reach a 50-year pavement life.

TYPES OF DOWEL BAR ALTERNATES

1. **Stainless steel clad alternates**
   - Stainless steel clad. These bars employ a patented manufacturing process that metallurgically bonds ordinary steel and stainless steel.
   - Stainless steel sleeves with an epoxy coated dowel bar insert. These bars have an epoxy-coated bar that is inserted into a thin walled stainless steel tube.

2. **Non-stainless, corrosion resistant alternates**
   - MMFX-2 steel dowel bars. These bars are high chromium but below the threshold to be classified as stainless. In addition, these bars have a dual phase steel microstructure that resists corrosion. Currently patented and manufactured by MMFX Steel Corporation (USA).

3. **Epoxy coated**
   - Epoxy coated. Traditional black steel bars with epoxy coating

Note: solid stainless bars are not recommended at this time due to their high initial cost. Other non-stainless, corrosion resistant bar alternatives are under investigation but are not yet approved for use. (These would include heavy galvanized steel bars and dual-phase steel bars).

APPLICATION OF DOWEL BAR TYPE SELECTION

1. Western Washington

   Preferred Dowel Bar Type: **Stainless steel clad alternates**
   Preferred Dowel Bar Spacing:
   - Truck Lanes (lanes 1 and 2 in multi-lane highways): Twelve dowel bars per joint
   - Non-truck lanes (Lanes 3, 4 or 5 in multi-lane highways): Eight dowel bars per joint (Four in each wheel path)
   - HOV lanes: Eight dowel bars per joint (Four in each wheel path)

   Note: the design for HOV lanes assumes these will remain as HOV lanes. The designer/engineer of record should carefully examine the potential future use of the HOV lanes to estimate the risk of this lane being converted to use by truck traffic. If there is a significant risk of the HOV lane being converted to a truck traffic lane, then a twelve dowel bars per joint configuration should be used.

2. Mountain Passes (greater than 2000 foot elevation)

   Preferred Dowel Bar Type: **Stainless steel clad alternates**

Preferred Dowel Bar Spacing:
- Truck Lanes (lanes 1 and 2 in multi-lane highways, and in lanes 1 and 2 in two lane sections): Twelve dowel bars per joint
- Non-truck lanes (Lanes 3, 4 or 5 in multi-lane highways): Eight dowel bars per joint (Four in each wheel path)

3. Eastern Washington

Preferred Dowel Bar Type: **Non-stainless, corrosion resistant alternates (MMFX-2)**
Preferred Dowel Bar Spacing:
- Truck Lanes (lanes 1 and 2 in multi-lane highways): Twelve dowel bars per joint
- Non-truck lanes (Lanes 3, 4, or 5 in multi-lane highways): Eight dowel bars per joint (Four in each wheel path)

4. Dowel Bar Retrofit (DBR) projects

Preferred Dowel Bar Type: Epoxy coated or non-stainless, corrosion resistant alternates. DBR projects are projected to have useful lives of about 15 years, reducing the need for highly corrosion resistant dowel bars. Epoxy coated bars have typically been used in DBR, but non-stainless, corrosion resistant bars could be allowed as an alternate. Dowel bar spacing remains three bars per wheel path.

5. Dowel Bar Specifications

Current dowel bar specifications and costs can be obtained from the State Materials Laboratory – Pavements Division.

APPENDIX 2 - WSDOT PROBABILISTIC INPUTS

Table A1.1. Input Probability Distributions Examples (FHWA).

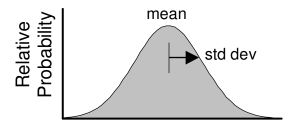
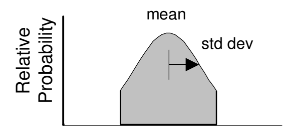
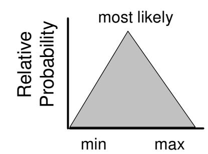
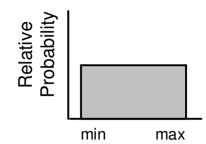
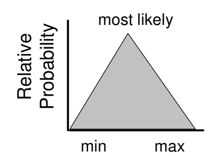
Distribution Type	Spreadsheet Formula	Illustration
Normal	lccanormal (mean, std dev)	
Truncated Normal	lccatnormal (mean, std dev, lower bound, upper bound)	
Triangular	lccatriang (minimum, most likely, maximum)	
Uniform	lccauniform (minimum, most likely, maximum)	
Triangular	lccatriang (minimum, most likely, maximum)	

Table A1.2. Project Details

Input	Unit
State Route	
Project Name	
Region	
County	
Analyzed By	
Begin MP	
End MP	
Lane Width	feet
Shoulder Width (left/right and inbound/outbound)	feet

Table A1.3. Analysis Options

Input	Unit	Probability Distribution	Value
Analysis Period	year	N/A	50
Discount Rate	%	Triangular	3, 4, 5
Beginning of Analysis Period		N/A	Year of Initial Construction
Include Agency Cost Residual Value		N/A	Yes
Include User Costs in Analysis		N/A	Yes
User Cost Comparison Method		N/A	Calculated
Traffic Direction		N/A	Both, Inbound or outbound
Include User Cost Residual Value		N/A	Yes

Table A1.4. Traffic Data

Input	Unit	Probability Distribution	Value
AADT (Both Directions) – Construction Year		N/A	Note 1
Single Unit Trucks as Percentage of AADT	%	N/A	Note 1
Combo Unit Trucks as Percentage of AADT	%	N/A	Note 1
Annual Growth Rate of Traffic	%	Normal	Note 1, 1.0
Speed Limit Under Normal Conditions	mph	N/A	Note 1
Lanes Open in Each Direction Under Normal Operation		N/A	Note 1
Free Flow Capacity	vphpl	Deterministic	Software provides calculator
Queue Dissipation Capacity	vphpl	Normal	1818, 144 (Note 2)
Maximum AADT Both Directions		N/A	Note 3
Maximum Queue Length	mile	N/A	Note 4
Rural/Urban		N/A	Note 1

Note 1 – Growth rate can be obtained from the WSPMS or through Regional information.

Note 2– observed flow rates (FHWA)

Note 3 – information contained in the Highway Capacity Manual

Note 4 – based on local experience

Table A1.5. Value of User Time

Input	Unit	Probability Distribution	Value
Value of Time for Passenger Cars	$	Triangular	12.00, 13.96, 16.00
Value of Time for Single Unit Trucks	$	Triangular	20.00, 22.34, 24.00
Value of Time for Combination Trucks	$	Triangular	25.00, 26.89, 29.00

Table A1.6. Traffic Hourly Distribution

Use default values contained in software program unless Region (or project) specific information is available.

Table A1.7. Added Vehicle Time and Cost

Use default values contained in the software program, unless Region (or project) specific information is available.

Table A1.8. Alternatives (initial and future rehabilitation)

Input	Unit	Probability Distribution	Value
Alternative Description		N/A	
Activity Description		N/A	
Agency Construction Cost	$1000	Normal	Cost, 10%
Activity Service Life	year	Triangular	Note 1
Maintenance Frequency	year	Triangular	Note 2
Maintenance Cost	$1000	Normal	Cost, 10%
Work Zone Length	mile	N/A	Value
Work Zone Capacity	vphpl	Deterministic	See Table A1.8.a
Work Zone Duration	days	Deterministic	Value
Work Zone Speed Limit	mph	N/A	Value
Number of Lanes Open in Each Direction During Work Zone		N/A	Value
Work Zone Hours		N/A	Value

Note 1: the minimum, most likely, and maximum expected life should be based on regional experience, data contained in the Washington State, and approved by the HQ Materials Laboratory – Pavements Division

Note 2: the minimum, most likely, and maximum expected life (if available) should be based on regional experience and approved by the HQ Materials Laboratory – Pavement Division

Table A1.8.a. Measured Average Work Zone Capacities [4].

Directional Lanes		Average Capacity	
Normal Operations	Work Zone Operations	Vehicles per Hour	Vehicles per Lane per Hour
3	1	1,170	1,170
2	1	1,340	1,340
5	2	2,740	1,370
4	2	2,960	1,480
3	2	2,980	1,490
4	3	4,560	1,520

APPENDIX 3 – PROBABILISTIC ANALYSIS EXAMPLE

This example is hypothetical. This project involves the removal and replacement of an existing interstate concrete pavement. Roadway configuration is 4 lanes in each direction with 10-foot right shoulders and 4-foot left shoulders. The alternatives evaluated will include:

1. Removal of the existing PCC and replacement with HMA
   a. Initial construction
      1.0 ft HMA
      <u>0.55 ft CSBC</u>
      1.55 ft total depth
   b. Initial construction thickness design based on 50-year performance with future overlays, 10 year cycle with minimum life of 6 years and maximum of 12 years
   c. Future Overlays
      i. 0.15 foot overlay in 1st, 3rd, and 5th cycles
      ii. 45 mm Mill and Fill in 2nd, 3rd, and 5th cycles

2. Removal of the existing PCC and replacement with PCC
   a. Initial construction
      1.0 ft HMA
      <u>0.55 ft CSBC</u>
      1.55 ft total depth
   b. Initial construction thickness design based on 50-year performance with future rehabilitation in 25th year
   c. Future Rehabilitation
      i. Diamond grinding to remove studded tire wear and reseal joints every 25 years (minimum of 20 years and maximum of 30 years)
      ii. 0.15 ft Mill and Fill in 2nd, 3rd, and 5th cycles (pavement life – minimum of 6 years, most likely 10 years, and maximum 12 years)

LCCA Input Data

1. Economic Variables	
Value of Time for Passenger Cars ($)	$11.50
	LCCATRIANG(10,11.5,13)
Value of Time for Single Unit Trucks ($)	$18.50
	LCCATRIANG(17,18.5,20)
Value of Time for Combination Trucks ($)	$22.50
	LCCATRIANG(21,22.5,24)

2. Analysis Options	
Include User Costs in Analysis	Yes
Include User Cost Residual Value	Yes
Use Differential User Costs	Yes
User Cost Computation Method	Calculated
Include Agency Cost Residual Value	Yes
Traffic Direction	Inbound
Analysis Period (Years)	60
Beginning of Analysis Period	2003
Discount Rate (%)	4.0
	LCCATRIANG(3,4,5)

3. Project Details and Quantity Calculations	
State Route	LCCA Example
Project Name	
Region	
County	
Analyzed By	L. M. Pierce
Beginning MP	0.00
Ending MP	5.00
Length of Project (miles)	5.00
Lane Width (ft)	12.00
	Right
Shoulder Width - Inbound (ft)	10.00
Shoulder Width - Outbound (ft)	10.00
Roadway Area (Square Feet)	1,584,000
Shoulder Area (Square Feet)	369,600
Total Area (Square Feet)	1,953,600

4. Traffic Data	
AADT (Both Directions) - Construction Year	200,000
Cars as Percentage of AADT (%)	90.0
Single Unit Trucks as Percentage of AADT (%)	3.0
Combination Trucks as Percentage of AADT (%)	7.0
Annual Growth Rate of Traffic (%)	2.5
	LCCANORMAL(2.5,2)
Speed Limit Under Normal Condition (mph)	65
No of Lanes in Each Direction During Normal Operation	5
Free Flow Capacity (vphpl)	2074
Rural/Urban	Urban
Queue Dissipation Capacity (vphpl)	1818
	LCCANORMAL(1818,144)
Maximum AADT (Both Directions)	400,000
Maximum Queue Length (miles)	10.0

Alternative 1

Initial Construction	Remove and Replace Existing PCCP with HMA	
Agency Construction Cost ($1000)	$12,686.00	
	LCCANORMAL(12686,1268.6)	
User Work Zone Costs ($1000)	$200.00	
Work Zone Duration (days)	165	
No of Lanes Open in Each Direction During Work Zone	3	
Activity Service Life (years)	9.3	
	LCCATRIANG(6,10,12)	
Maintenance Frequency (years)	0	
Agency Maintenance Cost ($1000)	0	
Work Zone Length (miles)	5.00	
Work Zone Speed Limit (mph)	35	
Work Zone Capacity (vphpl)	1500	
Time of Day of Lane Closures (use whole numbers based on a 24-hour clock)		
Inbound	Start	End
First period of lane closure	0	24
Second period of lane closure	0	0
Third period of lane closure	0	0
Outbound	Start	End
First period of lane closure	0	0
Second period of lane closure	0	0
Third period of lane closure	0	0

Rehabilitation #1	Mill and Fill with 2 inch HMA	
Agency Construction Cost ($1000)	$2,777.00	
	LCCANORMAL(2777,277.7)	
User Work Zone Costs ($1000)	$20.00	
Work Zone Duration (days)	25	
No of Lanes Open in Each Direction During Work Zone	4	
Activity Service Life (years)	9.3	
	LCCATRIANG(6,10,12)	
Maintenance Frequency (years)	0	
Agency Maintenance Cost ($1000)	0	
Work Zone Length (miles)	1.00	
Work Zone Speed Limit (mph)	35	
Work Zone Capacity (vphpl)	1500	
Time of Day of Lane Closures (use whole numbers based on a 24-hour clock)		
Inbound	Start	End
First period of lane closure	0	5
Second period of lane closure	21	24
Third period of lane closure	0	0
Outbound	Start	End
First period of lane closure	0	0
Second period of lane closure	0	0
Third period of lane closure	0	0

Rehabilitation #2	2 inch HMA Overlay	
Agency Construction Cost ($1000)	$3,409.00	
	LCCANORMAL(3409,340.9)	
User Work Zone Costs ($1000)	$30.00	
Work Zone Duration (days)	35	
No of Lanes Open in Each Direction During Work Zone	4	
Activity Service Life (years)	9.3	
	LCCATRIANG(6,10,12)	
Maintenance Frequency (years)	0	
Agency Maintenance Cost ($1000)	0	
Work Zone Length (miles)	1.00	
Work Zone Speed Limit (mph)	35	
Work Zone Capacity (vphpl)	1500	
Time of Day of Lane Closures (use whole numbers based on a 24-hour clock)		
Inbound	Start	End
First period of lane closure	0	5
Second period of lane closure	21	24
Third period of lane closure	0	0
Outbound	Start	End
First period of lane closure	0	0
Second period of lane closure	0	0
Third period of lane closure	0	0

Rehabilitation #3	Mill and Fill with 2 inch HMA	
Agency Construction Cost ($1000)	$2,777.00	
	LCCANORMAL(2777,277.7)	
User Work Zone Costs ($1000)	$20.00	
Work Zone Duration (days)	25	
No of Lanes Open in Each Direction During Work Zone	4	
Activity Service Life (years)	9.3	
	LCCATRIANG(6,10,12)	
Maintenance Frequency (years)	0	
Agency Maintenance Cost ($1000)	0	
Work Zone Length (miles)	1.00	
Work Zone Speed Limit (mph)	35	
Work Zone Capacity (vphpl)	1500	
Time of Day of Lane Closures (use whole numbers based on a 24-hour clock)		
Inbound	Start	End
First period of lane closure	0	5
Second period of lane closure	21	24
Third period of lane closure	0	0
Outbound	Start	End
First period of lane closure	0	0
Second period of lane closure	0	0
Third period of lane closure	0	0

Rehabilitation #4	2 inch HMA Overlay	
Agency Construction Cost ($1000)	$3,409.00	
	LCCANORMAL(3409,340.9)	
User Work Zone Costs ($1000)	$30.00	
Work Zone Duration (days)	35	
No of Lanes Open in Each Direction During Work Zone	4	
Activity Service Life (years)	9.3	
	LCCATRIANG(6,10,12)	
Maintenance Frequency (years)	0	
Agency Maintenance Cost ($1000)	0	
Work Zone Length (miles)	1.00	
Work Zone Speed Limit (mph)	35	
Work Zone Capacity (vphpl)	1500	
Time of Day of Lane Closures (use whole numbers based on a 24-hour clock)		
Inbound	Start	End
First period of lane closure	0	5
Second period of lane closure	21	24
Third period of lane closure	0	0
Outbound	Start	End
First period of lane closure	0	0
Second period of lane closure	0	0
Third period of lane closure	0	0

Rehabilitation #5	Mill and Fill with 2 inch HMA	
Agency Construction Cost ($1000)	$2,777.00	
	LCCANORMAL(2777,277.7)	
User Work Zone Costs ($1000)	$20.00	
Work Zone Duration (days)	25	
No of Lanes Open in Each Direction During Work Zone	4	
Activity Service Life (years)	9.3	
	LCCATRIANG(6,10,12)	
Maintenance Frequency (years)	0	
Agency Maintenance Cost ($1000)	0	
Work Zone Length (miles)	1.00	
Work Zone Speed Limit (mph)	35	
Work Zone Capacity (vphpl)	1500	
Time of Day of Lane Closures (use whole numbers based on a 24-hour clock)		
Inbound	Start	End
First period of lane closure	0	5
Second period of lane closure	21	24
Third period of lane closure	0	0
Outbound	Start	End
First period of lane closure	0	0
Second period of lane closure	0	0
Third period of lane closure	0	0

Rehabilitation #6	2 inch HMA Overlay	
Agency Construction Cost ($1000)	$3,409.00	
	LCCANORMAL(3409,340.9)	
User Work Zone Costs ($1000)	$30.00	
Work Zone Duration (days)	35	
No of Lanes Open in Each Direction During Work Zone	4	
Activity Service Life (years)	9.3	
	LCCATRIANG(6,10,12)	
Maintenance Frequency (years)	0	
Agency Maintenance Cost ($1000)	0	
Work Zone Length (miles)	1.00	
Work Zone Speed Limit (mph)	35	
Work Zone Capacity (vphpl)	1500	
Time of Day of Lane Closures (use whole numbers based on a 24-hour clock)		
Inbound	Start	End
First period of lane closure	0	5
Second period of lane closure	21	24
Third period of lane closure	0	0
Outbound	Start	End
First period of lane closure	0	0
Second period of lane closure	0	0
Third period of lane closure	0	0

Alternative 2

Initial Construction	Remove and Replace Existing PCC with PCC	
Agency Construction Cost ($1000)	$18,249.00	
User Work Zone Costs ($1000)	$300.00	
Work Zone Duration (days)	165	
No of Lanes Open in Each Direction During Work Zone	3	
Activity Service Life (years)	35.0	
	LCCATRIANG(25,35,45)	
Maintenance Frequency (years)	0	
Agency Maintenance Cost ($1000)	0	
Work Zone Length (miles)	5.00	
Work Zone Speed Limit (mph)	35	
Work Zone Capacity (vphpl)	1500	
Time of Day of Lane Closures (use whole numbers based on a 24-hour clock)		
Inbound	Start	End
First period of lane closure	0	24
Second period of lane closure	0	0
Third period of lane closure	0	0
Outbound	Start	End
First period of lane closure	0	0
Second period of lane closure	0	0
Third period of lane closure	0	0

Rehabilitation #1	Diamond Grinding and Joint Resealing	
Agency Construction Cost ($1000)	$2,441.00	
	LCCANORMAL(2441,244.1)	
User Work Zone Costs ($1000)	$50.00	
Work Zone Duration (days)	50	
No of Lanes Open in Each Direction During Work Zone	4	
Activity Service Life (years)	15.0	
	LCCATRIANG(10,15,20)	
Maintenance Frequency (years)	0	
Agency Maintenance Cost ($1000)	0	
Work Zone Length (miles)	1.00	
Work Zone Speed Limit (mph)	35	
Work Zone Capacity (vphpl)	1500	
Time of Day of Lane Closures (use whole numbers based on a 24-hour clock)		
Inbound	Start	End
First period of lane closure	0	5
Second period of lane closure	21	24
Third period of lane closure	0	0
Outbound	Start	End
First period of lane closure	0	0
Second period of lane closure	0	0
Third period of lane closure	0	0

Rehabilitation #2	Diamond Grinding and Joint Resealing	
Agency Construction Cost ($1000)	$2,441.00	
	LCCANORMAL(2441,244.1)	
User Work Zone Costs ($1000)	$50.00	
Work Zone Duration (days)	50	
No of Lanes Open in Each Direction During Work Zone	4	
Activity Service Life (years)	15.0	
	LCCATRIANG(10,15,20)	
Maintenance Frequency (years)	0	
Agency Maintenance Cost ($1000)	0	
Work Zone Length (miles)	1.00	
Work Zone Speed Limit (mph)	35	
Work Zone Capacity (vphpl)	1500	
Time of Day of Lane Closures (use whole numbers based on a 24-hour clock)		
Inbound	Start	End
First period of lane closure	0	5
Second period of lane closure	21	24
Third period of lane closure	0	0
Outbound	Start	End
First period of lane closure	0	0
Second period of lane closure	0	0
Third period of lane closure	0	0

Rehabilitation #3		
Agency Construction Cost ($1000)		
User Work Zone Costs ($1000)	$50.00	
Work Zone Duration (days)		
No of Lanes Open in Each Direction During Work Zone	4	
Activity Service Life (years)	15.0	
Maintenance Frequency (years)	0	
Agency Maintenance Cost ($1000)	0	
Work Zone Length (miles)	1.00	
Work Zone Speed Limit (mph)	35	
Work Zone Capacity (vphpl)	1500	
Time of Day of Lane Closures (use whole numbers based on a 24-hour clock)		
Inbound	Start	End
First period of lane closure		
Second period of lane closure		
Third period of lane closure		
Outbound	Start	End
First period of lane closure		
Second period of lane closure		
Third period of lane closure		

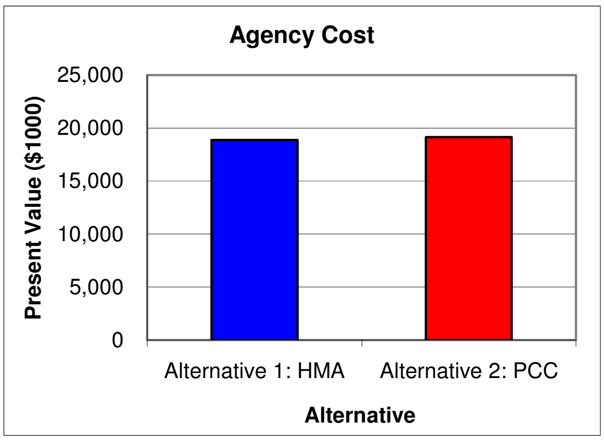
Rehabilitation #4		
Agency Construction Cost ($1000)		
User Work Zone Costs ($1000)	$50.00	
Work Zone Duration (days)		
No of Lanes Open in Each Direction During Work Zone	4	
Activity Service Life (years)	15.0	
	LCCATRIANG(10,15,20)	
Maintenance Frequency (years)		
Agency Maintenance Cost ($1000)		
Work Zone Length (miles)	1.00	
Work Zone Speed Limit (mph)	35	
Work Zone Capacity (vphpl)	1500	
Time of Day of Lane Closures (use whole numbers based on a 24-hour clock)		
Inbound	Start	End
First period of lane closure		
Second period of lane closure		
Third period of lane closure		
Outbound	Start	End
First period of lane closure		
Second period of lane closure		
Third period of lane closure		

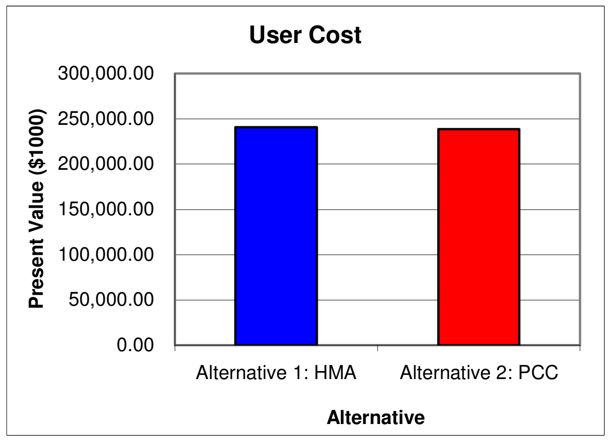
Rehabilitation #5		
Agency Construction Cost ($1000)		
User Work Zone Costs ($1000)	$50.00	
Work Zone Duration (days)		
No of Lanes Open in Each Direction During Work Zone	4	
Activity Service Life (years)	15.0	
	LCCATRIANG(10,15,20)	
Maintenance Frequency (years)		
Agency Maintenance Cost ($1000)		
Work Zone Length (miles)	1.00	
Work Zone Speed Limit (mph)	35	
Work Zone Capacity (vphpl)	1500	
Time of Day of Lane Closures (use whole numbers based on a 24-hour clock)		
Inbound	Start	End
First period of lane closure		
Second period of lane closure		
Third period of lane closure		
Outbound	Start	End
First period of lane closure		
Second period of lane closure		
Third period of lane closure		

Rehabilitation #6		
Agency Construction Cost ($1000)		
User Work Zone Costs ($1000)	$50.00	
Work Zone Duration (days)		
No of Lanes Open in Each Direction During Work Zone	4	
Activity Service Life (years)	15.0	
	LCCATRIANG(10,15,20)	
Maintenance Frequency (years)		
Agency Maintenance Cost ($1000)		
Work Zone Length (miles)	1.00	
Work Zone Speed Limit (mph)	35	
Work Zone Capacity (vphpl)	1500	
Time of Day of Lane Closures (use whole numbers based on a 24-hour clock)		
Inbound	Start	End
First period of lane closure		
Second period of lane closure		
Third period of lane closure		
Outbound	Start	End
First period of lane closure		
Second period of lane closure		
Third period of lane closure		

Deterministic Results

Total Cost	Alternative 1: HMA		Alternative 2: PCC	
	Agency Cost ($1000)	User Cost ($1000)	Agency Cost ($1000)	User Cost ($1000)
Nominal $	$30,107.67	$270,356.78	$22,317.33	$261,385.30
Present Value	$18,891.08	$240,884.78	$19,133.72	$238,485.30
EUAC	$835.02	$10,647.55	$845.75	$10,541.49
Lowest Present Value Agency Cost			Alternative 1: HMA	
Lowest Present Value User Cost			Alternative 2: PCC	



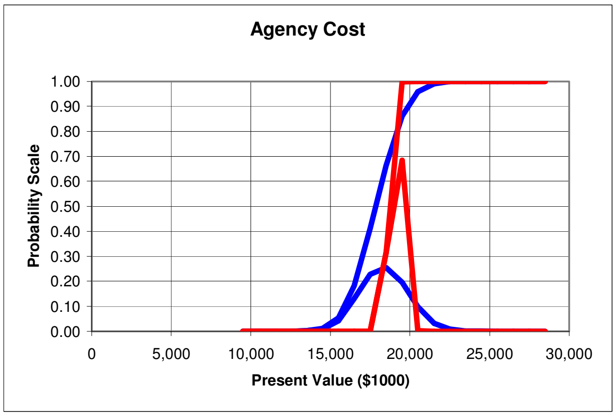


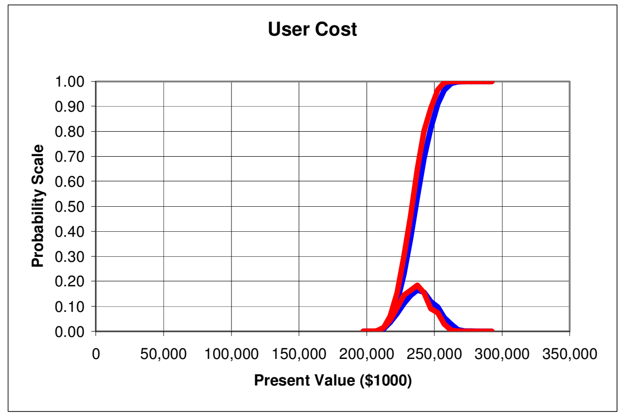
Based on the deterministic analysis, the PCC alternative is slightly higher for the present value of agency costs (approximately 1.3 percent higher) than the HMA alternative. For the present value of user costs, the PCC alternative is slightly lower (approximately 1.00 percent lower) than the HMA alternative. Based on total present value costs, these two alternatives would be considered equivalent (PCC is approximately 0.8 percent lower than HMA).

Probabilistic Results

Total Cost (Present Value)	Alternative 1: HMA		Alternative 2: PCC	
	Agency Cost ($1000)	User Cost ($1000)	Agency Cost ($1000)	User Cost ($1000)
Mean	$18,365.23	$239,105.37	$19,153.28	$236,004.03
Standard Deviation	$1,511.70	$11,478.88	$271.21	$10,363.83
Minimum	$13,083.68	$209,885.91	$18,553.22	$209,779.17
Maximum	$24,641.06	$275,664.34	$20,249.69	$265,385.41

Total Cost (Present Value)	Alternative 1: HMA		Alternative 2: PCC	
	Agency Cost ($1000)	User Cost ($1000)	Agency Cost ($1000)	User Cost ($1000)
Mean	$18,365.23	$239,105.37	$19,153.28	$236,004.03
Standard Deviation	$1,511.70	$11,478.88	$271.21	$10,363.83
Minimum	$13,083.68	$209,885.91	$18,553.22	$209,779.17
Maximum	$24,641.06	$275,664.34	$20,249.69	$265,385.41





Based on the cumulative probability distributions shown above, there is an 80 percent probability that the agency costs for the HMA alternative will be less than the PCC alternative. The above graph also shows that there is a lower risk of cost variation with the PCC alternative. The slopes of the cumulative risk profiles shown above are similar for the user costs and only a slight difference for the agency costs. The alternative with the steeper slope would have less variability and, the means being similar, would also be the preferred alternative.

APPENDIX 4 – POSSIBLE ENGINEERING ANALYSIS ITEMS FOR CONSIDERATION

- Air pollution impacts. Consider if either effects on traffic or effects during production affect the project or future preservation efforts
- Non-user impacts. How are surrounding neighborhoods affected by the project? How do these impacts vary depending on the type of pavement selected? What are the impacts at the point of production?
- Haul routes through neighborhoods. Consider the impacts both during initial construction and future preservation projects.
- Future ability of plants to operate at night in urban areas and associated cost increases. Where are typical production plants located? Will night work continue to be feasible in the area of plant production or will urban growth affect this? What possible effect will urban growth have on making production plants move further away from the corridor?
- Neighborhood impacts due to trip diversion during preservation projects. When a highway closure impacts the traveling public, many will divert to other routes to avoid delays. These diversions have associated costs, in and of themselves. Some of the costs come from backups and delays (user impacts) on these diversion routes; some of the costs come from impacts to neighborhoods, through increased traffic, noise, congestion, air pollution, safety and accident risks. Consider the level of user delays and the likelihood that diversions will occur and the level of impact these diversions could have on non-highway users.
- Business impacts due to reduced or restricted access. This impact happens both due to direct impacts to users and to impacts due to diversion. The magnitude grows as an area urbanizes and increases the number of businesses that stay open for extended (mostly nighttime) hours of operation. Diversion through a neighborhood with extensive commercial business can greatly impact those businesses.
- Effect of nighttime noise variances and risk of approval of noise variances. These two items tie in with the item noted above. As urban areas grow, nighttime noise variances become more difficult to obtain and more restrictive in their limitations. Review the corridor in question and the expected growth projections, to develop an idea of the risk associated with this non-user impact. Noise restrictions can limit hours of operations to the point of preventing work, or they can restrict noise levels below that achievable by state of the practice construction equipment. Noise restrictions apply also to vibration and noise generated by vibratory equipment and these restrictions can prevent the use of particular equipment within selected urban corridors. A single resident affected by nighttime noise can and has effectively shut down projects, forcing a move to day work and created huge impacts on highway users through delay and impacts.
- Noise
  - Pavement noise. Pavement surface texture can have an effect on noise through a corridor with some pavement surfaces being measurably quieter than others
  - Noise walls. Evaluate whether the corridor already has noise walls or is expected to have noise walls by the time of the project, and the impacts having or not having the walls might have on non-users/residents, both for construction noise and pavement noise
  - Haul through neighborhoods, at night. Haul though neighborhoods at night (and if you are hauling, you are traveling through *someone's* neighborhood) should be considered as an impact. Sparsely populated areas will obviously have a smaller impact due to the noise of hauling vehicles than will densely packed urban areas.
  - Noise from diverted traffic and other impacts. Diverted traffic must drive through someone's neighborhood to get to where they are going. At night, diverted traffic, especially involving large trucks, can have a significant noise impact on neighborhoods.
  - Noise generation during preservation projects. When preservation projects are performed on any pavement, noise is generated and its impacts on the local community must also be considered, in addition to the impact of the noise from the initial construction.
- Safety.
  - Public exposure to traffic control during lane closures.
  - Work exposure to traffic during lane closures.
  - Lane closures are a safety risk factor for both workers and the traveling public. Limited vision, nighttime lighting, temporary traffic control and other factors increase the risk of accidents to both

motorists and to workers within the work zone. Evaluate the risk to both, given the nature of the corridor, the ADT, the degree of urbanization and the complexity of the facility.
  - Safety risks associated with maintenance by state forces between preservation projects.
- Pavement type continuity within a corridor (similar to architectural choices for structures and wall-types within a corridor and landscape architectural choices for continuity within a corridor). It is generally not desirable to switch pavement types over relatively short stretches of highway. Maintenance needs change for each given pavement type, as do preservation needs. Further, the change in pavement type impacts the public in various ways, including aesthetics.
- Environmental effects.
  - Runoff temperature due to heating effects depending on pavement type. Evaluate in conjunction with design of the storm sewer system.

APPENDIX 5 – WSDOT PAVEMENT TYPE SELECTION COMMITTEE CORRESPONDENCES

Washington State Department of Transportation

Memorandum

June 29, 2004

TO: Don Nelson, 47321

FROM: Tom Baker, 47365

SUBJECT: Pavement Type Selection Protocol

When the pavement type selection has been completed and forwarded to the HQ Materials Laboratory, the Pavement Division will formulate the Pavement Type Selection Committee (referred to as the Committee) Approval Letter and request that each member of the Committee sign and forward the letter on to the next member. The Committee is not required to convene if the life cycle cost analysis between the alternatives is greater than 15 percent and the recommendations are acceptable to both the Region and the HQ Materials Laboratory. The Approval Letter shall provide the necessary documentation that supports the Committee's selection of the pavement type.

Projects to be reviewed shall be distributed to the Committee members for approval (see attached example of Approval Letter). Based on this review and obtaining consensus from the Committee, the Pavement Division will either process the Approval Letter, take appropriate action to obtain consensus, or convene the Committee.

In order to expedite the required time and expended level of effort for the review of pavement type selection projects, the following procedure is recommended:

1. The Committee should convene if the pavement type recommended by the Region is contrary to pavement design and engineering analysis recommendations. The pavement design and engineering analysis recommendations shall be subject to the review of the Pavement Division or any member of the Committee. Under these circumstances it shall be the responsibility of the Pavements Division or the Committee member to formulate, in writing, why the selected pavement type is not appropriate and distribute his/her rationale to all members. If all members agree with the recommendations a meeting will not be necessary, otherwise, the Committee should convene.

2. The Committee should convene at the request of any member.

TEB:lmp

Washington State Department of Transportation

Memorandum

PAVEMENT TYPE SELECTION

SR-3
Luoto Road to SR-305
MP 48.90 to MP 53.00

The Pavement Type Selection Committee has completed its review of the pavement type selection for the project SR-3 Luoto Road to SR-305, MP 48.90 to MP 53.00.

The project consists of constructing the final two lanes of the ultimate four-lane facility from Luoto Road to SR-305.

The pavement design analysis resulted in both pavement types (HMA and PCC) being viable. In the life cycle cost analysis, one PCC alternative was compared to one HMA alternative. In the life cycle cost analysis of the two alternatives, there is a cost advantage in the use of HMA over PCC of greater than 15 percent. The Committee approves the use of HMA on this project.

The Pavement Type Selection Committee

Don Nelson ________________________
Director, Environmental and Engineering

Harold Peterfeso ________________________
State Design Engineer

John Conrad ________________________
Assistant Secretary
Engineering and Regional Operations

Greg Selstead ________________________
Director of Project Control and Reporting

Tom Baker ________________________
State Materials Engineer

Randy Hain ________________________
Olympic Region Administrator

LMP:bg